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Design of Mill Buildings The literature on steel mill-building design becomes more extensive every year. Much of it deals with methods of stress analysis and considerations more or less theoretical. In the article on page 467 Mr. Fleming has purposely avoided those matters which are covered in the books and has presented a résumé of those features which are less often recorded, but no less important to the designer. Necessarily, completeness demanded that many things be given which are well known to the experienced designer. Scattered through the article will be found bits of information and generalizations which are the lessons of a lifetime devoted to steel-work design of the greatest possible variety. These will make the article as valuable to the experienced designer as its completeness renders it helpful to the young engineer.

International Congress The issuance of a preliminary prospectus outlining the scope of the International Engineering Congress to be held in San Francisco in September, 1915, in connection with the Panama-Pacific Exposition, indicates that though the first steps in bringing about an agreement as to the congress among the five national societies were very slow, the committee of management is losing no time in perfecting the arrangements. Even now little more than a skeleton is prepared, but the indications are that the detailed work is well in hand. It is patent that an undertaking of this sort will need wide support from American engineers in order to be a success, and there should be generous response to the committee's circular on membership. To those who expect to see the exposition the returns technically and in acquaintances made will be large in comparison with the trifling outlay in membership fee.

The English Channel Tunnel That the latest English Channel tunnel proposal, briefly noted in this journal some time ago, is not allowed to slumber is evident from the attention given it in the British press. With the change in attitude of the Government from one of strong opposition to willingness to consider the proposal with care and sympathy has come the hope that the stupendous project will be realized. Basing estimates upon present cross-channel traffic, both freight and passenger, the promoters expect that the annual profit on completion of the tubes would be £1,135,000, while the capital required would be £16,000,000. As a financial venture, therefore, if the figures are reliable, the scheme gives promise of success. The technical features, as now developed, were given recently in an address by Sir Francis Fox, of Sir Douglas Fox & Partners. The tubes would be 18 ft. in net internal diameter placed 36 ft. center to center, with cross-passages at intervals. They would be driven through the lower, or gray, chalk, a very homogeneous material, practically free from, and almost impervious to, water. It possesses, moreover, the peculiar property of gradually puddling itself. A drainage heading, 11 ft. in diameter, would be driven, which, at the rates assumed, would require four years to complete. The main tunnels might or might not be driven at the same time that the headings are. The total length would be about 31 miles, of which 24 would be under water. No definite announcement of the British Government's attitude has yet been made, and until then the scheme will not pass out of academic bounds.

Brick-Pavement Cushions Probably the most interesting feature of the pavement test at the Road Congress, referred to editorially in this issue on page 450, was the contrast between the life of the two sections of brick pavement. They were alike in all respects—foundation, make of brick, kind of filler and date of laying—save only that in the one that endured the cushion was a 1:5 mortar, while in the other a plain sand cushion was used. Why the one should have stood up so much better than the other is difficult to understand. Unquestionably the mortar bed tended to make the entire pavement—concrete base, cushion and brick—a thick monolith. Nevertheless one would expect the grout filler to hold the brick so rigidly that the difference in cushions would have but small influence. Anyone who has seen the wonderful single-course brick pavement standing as a 6-ft. cantilever, where a flood undermined the sub-base and tore out the concrete foundation, near Cleveland, would certainly expect the grout filler to render the cushion of less importance than was indicated in this test. Certainly the result warrants a careful trial of the lean-mortar cushion in practice.

A Successful Departure It is a pleasure to record the success which attended the first attempt of the American Society of Civil Engineers to hold its mid-month meeting elsewhere than in New York City. An attendance of 450 was registered at New Orleans, and of the 125 members of the society present twenty-one came from New England and the Central States, while there were representatives from points as far west as the coast and as far north as Winnipeg. The business meetings were noticeably well attended and much interest was manifested in the affairs of the society. The meeting was an experiment. It was a success and should lead to the holding of similar meetings in other parts of the country. It is well to point out, however, that unless there is the enthusiasm and interest displayed by the New Orleans and Louisiana members future meetings will not be equally successful. The members in District 6 worked hard to acquaint the members of the society with the hospitality they had prepared. They have deserved well of the society and, at the same time, have set a pace for other localities which aspire to have the interim meetings within their doors.

Imhoff Tank Design There has been no phase of the sewage-treatment problem during the past few years which has excited more interest among sanitary engineers in this country than the introduction of Dr. Imhoff's two-compartment settling tanks. The advantages of a form of sedimentation providing for the separation of the incoming sewage from the deposited sludge won almost instant recognition, and the number of such plants now in operation in the United States attests their popularity. The simplicity of these tanks is apt to create the belief that any engineer can design one properly. A little knowledge is a dangerous thing, however, and the engineer without special training who dons the mantle of the sanitary expert is apt to find his new cloak somewhat roomy. For seven years the engineers of the Emscher River Board in Germany have been engaged in the tank treatment of sewage, and their experience, as summed up in a series of three articles by Mr. Frank and Mr. Fries, of Dr. Imhoff's technical staff, should prove of unusual value to the sanitary profession in the United States. The first of these articles, dealing with fundamental considerations in the design of treatment plants and suggestions relating to the selection of preliminary screens and the proper proportions for grit chambers, appears on page 452 of this issue. The second article will discuss the settling compartment, its width and depth with respect to the proper period of detention of the sewage, the relative advantages of radial and horizontal flow, velocity of flow, the proper and improper use of baffles and scum boards and inlet and outlet details. The third article will be devoted to the sludge-digestion compartment and to methods of disposing of sludge. These are the very things which an engineer must know. Imhoff-tank data in this country have been rather meager, so that those who would profit by the experiences in the Emscher District would do well to give the articles by Mr. Frank and Mr. Fries careful study.

An Ounce of Prevention As old as civilization itself is the principle of looking back over the experiences of others in order to perfect one's plans for the future. Yet the engineer often finds it hard to get the board of laymen which he may happen to serve to realize that the principle which applies to the individual or a business is applicable also to municipal engineering practice. Just here is involved one of the highest functions of the engineer. In advising the council of the growing city he needs keen insight and vision, for while he must not sanction over-development, he must take a determined stand for a broad view of the questions in hand, and more than likely he will have to convert to this view men who have not had the opportunity of studying development questions in their widest aspects. This is particularly true in the laying out of sewerage systems. Too often it is found that large sums have been spent on systems which, when the population has increased, require expensive auxiliary lines. Judging from the records of many large cities one might be inclined to ask whether it were not usually feasible to design a large system, so that it would not overburden the taxpayer of to-day and yet permit of great expansion without undue additional expense. It certainly is possible in some cases to design a system in this manner. The one described on page 472 of this issue has been laid out with the ultimate development of the city in mind, and yet carries no unreasonable first cost. The most advantageous system has been laid out for the entire area which the city and adjacent municipalities are fairly sure to cover, though only those parts will be built which are needed at present, the first work being on the trunk sewers. In most cases the expensive interceptors and outfalls are left for later construction, temporary discharge being provided into waters now available for sewage dilution, but which eventually will be unable to care for it. The significance of the situation lies in the fact that the sewers are so located that interceptors can take the place of the temporary outfalls whenever increased service demands it, and, incidentally, provides the money for extension.

Accelerated Pavement Tests

As recounted elsewhere in this issue, accelerated tests were made of sample concrete and brick pavements at the Road Congress in Detroit. Two iron-shod wheels, reinforced with hammers simulating a horse's shoe, ground mercilessly for six days with the results shown in the photographs on pages 457 and 458. Some thoughts inspired by the ordeal are worthy of comment. The plain facts are that two of the four classes emerged from the test good pavements and that two were destroyed. The granite concrete and the brick set on a mortar cushion wore evenly and were in good condition; the gravel concrete had about given up the ghost, while the brick on a sand cushion had passed into the "bad-pavement" class. Comparisons, if any, should be made between the two that failed and the two that passed; in both cases the brick had the better of the argument. Quantitative comparisons of wear will be possible when the level readings taken by members of the civil engineering faculty of the University of Michigan become available. But having stated the conclusions in general it is pertinent to add some observations on the significance of the tests.

Frankly the Engineering Record believes that the public demonstration of the determinator was a mistake. It is well understood by investigators that accelerated tests have a place in determining the value of engineering materials and equipment, though close study and excellent judgment are needed for translating them into service values. Even then they must be taken with suspicion, and as to many materials, competent specialists agree that they give no rational indication of prospective value in service. The place, then, for accelerated tests is not in public, but in the calm, judicial atmosphere of the laboratory. Even granting that the determinator is more rational than the average accelerated test, its exhibition where laymen were invited to draw their own conclusions, without being furnished with the "conversion factors," so to speak, to enable them to estimate what these results meant when translated into pavement service, was a violation of good engineering sense.

It may be argued that on account of the fortuitous (?) outcome—one pavement of each principal type standing up and one failing—that at the worst the laymen could only conclude that some brick and some concrete pavements fail. But, first, this lacks definiteness and tells nothing not already known, and, secondly, is grossly erroneous, insofar as it condemns pavements which, when laid as well in practice as these were, have given excellent results. In Cleveland and Cuyahoga County, Ohio, will be found miles of first-class brick pavement laid on a sand cushion, while hardly a stone's throw from the testing machine were highways that gave the lie to the adverse conclusions that the layman was sure to draw from the gravel-concrete test section. Made with the same aggregate and the same mix, and laid by the same forces, the one in service must receive a favorable verdict and the other be condemned. Of what value, then, is the test?

Hidden from those whose judgment, even with the best of intentions, necessarily had to be "snap judgment," the determinator may prove a valuable instrument in guiding men of trained judgment in the improvement of pavements. Operated on the highways, so that all who run may read, it is a snare and a delusion.

Cedars Rapids Power Plant

If anyone had ventured the prediction two years ago that the then unprecedented dimensions of the Keokuk wheels would shortly be exceeded his remarks would have been received with a smile of incredulity. Yet even before those wheels were in operation plans for larger units had been perfected. The wheels at Cedars Rapids, described on page 461, are 25 per cent heavier than those at Keokuk, and their power output is 8 per cent greater. One limit was reached at Keokuk, however—the shipping size of a single-piece unit—and all the new wheels will be cast in four pieces, as were some of those for the Mississippi River installation. The enormous size of the wheels is by no means the only item of interest in connection with the 160,000-hp installation of the Cedars Rapids Manufacturing & Power Company. The wheel setting and the governor system are worthy of special comment.

At Cedars Rapids the speed ring of the wheel takes care of the load and carries it to the concrete around and below the draft tube, and, furthermore, the thrust bearing is above the generator. Just what this means in first cost can best be appreciated by comparing this setting with that at Keokuk. There the wheel is suspended from a pit liner, the upper part of which is fitted to a floor ring built into the substructure. In order to take care of such heavy loads a little below the floor plane, there was constructed a strong concrete structure, 16 ft. deep, above the wheel chamber. At Cedars Rapids, on the other hand, this section needs to be only 7¾ ft. deep, pointing to a possible saving in the 100 x 1600-ft. power house at Keokuk of about 49,000 cu. yd. of concrete, amounting, at $5.50 per yard, to $270,000.

The governor system has very decided advantages, and, in fact, is one of the best so far designed. The concentration of all pumping equipment, necessitating only a tank for each unit, brings the maintenance and operating costs to a minimum, while the use of a central system eliminates much of the noise found in plants where each unit has a separate pump. The design of the accumulator tanks for each unit is also worthy of note. The loading of such tanks with weights is not good practice, though frequently done. When the fluid—oil or water—is required by the governor there is a sudden demand and the inertia of the weights prevents proper response. Here, however, the necessary pressure is secured by compressed air, giving an accumulator superior to any other in responsiveness.

One feature of the governor system, however, could well be improved. The water is returned to the pumps in open concrete channels. As this water is perfectly pure, being secured from condensers, it will have a solvent action on the concrete, with consequent danger of deposition of the dissolved elements in the delicate parts of the governor. A metal lining in the conduits or return pipes would be advisable.

An examination of the details of this installation will disclose other points of interest to hydraulic engineers; but those noted seem to deserve special mention.

Hydrological Dangers

The history of the reclamation work in the Florida Everglades, which is briefly outlined on page 454, is like that of many irrigation projects, except that the troubles have been due to too much rather than to too little water. It will be difficult to discover a more emphatic lesson of the importance of a thorough study of rainfall and runoff in drainage and irrigation than is afforded by the work upon which the State of Florida has embarked. It is evident that the canal system which is under construction there is wholly inadequate for its purposes, and it is equally evident that its deficient capacity is due to inadequate preliminary engineering investigations. Drainage undertakings are assuming such importance in several sections of the country that the Florida lesson ought to be well learned by the engineering profession.

The distribution of rainfall over large areas is something that the experienced engineer rarely guesses about. It is affected by so many conditions that there is no scientific approximation to actual records which is usually sufficiently accurate to warrant heavy investments upon its results. This is particularly true in districts where the local conditions are unusual. The transfer of the average records of one district to meet the conditions of another with different topographical and climatic characteristics invites disaster.

The interpretation of accurate hydrological records is an undertaking of no small proportion. In the Florida case Messrs. Mead, Hazen and Metcalf report that not only were available records not used in preparing the plans for the drainage of the Everglades and the regulation of Lake Okeechobee, but inapplicable records were employed in part, and the deductions as to the probable runoff were based on unsound reasoning. Furthermore, it is their opinion that the uncertainties involved in the basic data, upon which the plans were predicated, were not clearly brought out in the report on them, and the reader was left to infer that the conclusions were drawn from sufficiently accurate data for the development of this great project, involving the expenditure of millions of dollars.

The result of this engineering blunder has been to involve the State of Florida in something approaching a land scandal, to bring the men engaged in a legitimate real estate development into a most disagreeable position, and to lead many thousands of people to invest in swamp land in the belief that it was ready for market gardening. It is a rather striking commentary on the relative business methods of public and private bodies that the first thorough preliminary study of the Everglades problem was made for the private land company which is now going ahead to protect its lands and those it has sold according to the plans of its engineering advisers. Later, the State, awakened to the situation, determined to follow the company's example. These unreclaimed lands are a valuable public asset, and the State must obtain reliable information concerning the technical aspects of the work of reclamation. Until this is secured the scheme is like a ship without a chart, likely to run aground on many hidden shoals.

Congested Freight Terminals

The lack of proper balance or adjustment among the various instrumentalities of transportation is strikingly illustrated by the frequent congestions which occur in freight terminals of large traffic centers. Chicago in 1907 suffered from a freight congestion the effects of which were felt for years; in fact, it caused traffic managers of large commercial houses at other points to avoid routing their through traffic via the Chicago gateway. The Chicago railroads have, as a consequence, lost millions of dollars in freight revenue by the failure to remedy conditions and to provide adequate freight interchange facilities which would enable through traffic to pass without delay.

The usual movement of full carload traffic through Chicago from one carrier to another requires from three to six days. Less than carload traffic requires from two to six days, and to avoid serious delay a large part of this class of freight is drayed from one terminal to another at heavy cost to the railways. The drayage across the congested city, causing still further congestion in street traffic, frequently costs as much as the rail haul accruing to the railway.

Millions upon millions of dollars have been expended in union passenger terminals, but little effort has been made to provide adequate union freight terminals, where freight traffic could be expeditiously, efficiently and economically handled. Recently an attempt has been made at Chicago to provide facilities for interchanging freight among a certain number of roads, but because of the failure to make the facilities large enough to locate them properly and admit all the railroads entering the city it will not prove successful, except in a minor degree, since the roads not embraced in the plan will continue to interchange their traffic in the same slow and expensive manner.

At Buffalo, as described in this journal last week, it has been planned to provide facilities where all roads may freely interchange traffic. The fact that the principal roads concerned have committed themselves to independent policies and have spent large sums carrying them out precludes the likelihood of anything coming of the scheme. The idea is the correct one, however, and would result in securing for the Buffalo railways a vast tonnage which would otherwise seek other channels. "Traffic must and will seek the lines of least resistance," and the railway managers of Chicago lines, if awakened to a proper sense of their responsibilities, will provide adequate means for the prompt and economical interchange of their through traffic.

That the proper balance has not been maintained in the increase in equipment and yards and terminals is proved by statistics for the ten-year period from 1903 to 1912.

In the nine years ended with 1912 the tractive power of locomotives increased 109.6 per cent, and in the ten years ended with the same year the total capacity of freight cars increased 95 per cent. On the other hand, in the ten-year period the capacity of yard tracks and sidings increased only 74 per cent. In other words, the terminal facilities for handling the increased traffic are about one-third out of balance, and to keep pace with the traffic terminal facilities should be increased about 33 per cent before any additional equipment is purchased. Whenever 1000 new freight cars are built it requires about 8 miles of side tracks to accommodate them. The failure to provide these tracks causes congestion and reduces the efficiency of new and old equipment.

A Failure in Appraisals

The local row in Chicago over garbage disposal is one of the bitterest witnessed in recent years in that city, as those who have followed the progress of the negotiations through the columns of this journal realize. As emphasized in this journal, it is not good sanitary practice to dump garbage in an abandoned clay pit in a residential district, even if the refuse is given chemical treatment. But these features of the situation, interesting though they may be, are not so instructive to the engineer as the history of the attempt to appraise the property of the Chicago Reduction Company. Appraisal work is rapidly becoming an important engineering specialty and a case where precedent in such work has been kicked aside, as in this instance, is worth more than casual mention. An abstract of the opinions of the appraisers will be found on page 465.

The agreement for appraisal between the company and the city stated: "The said City of Chicago desires to purchase the plant of the said The Chicago Reduction Company for the purpose of operating the same to dispose of the city's garbage and The Chicago Reduction Company has agreed to sell the same." This statement, signed by both parties, shows clearly that the city and the company were willing buyer and seller respectively, and the value of the property should be determined by the appraisers on that basis and not on the basis of a condemnation to get the plant out of the way so that its site could be used for another purpose.

This mutual understanding is still further shown by the following clause in the agreement made by the city to buy and the company to sell the plant: "The appraisal is to be made upon the plant as a going concern, taking into consideration the reasonable development expenses incurred by said company in bringing said garbage-reduction plant to its present condition, with all of the natural accretions thereto, and is to include the lands upon which said plant is now located, taking into consideration the peculiar adaptability for the purposes for which it is now being used, with all the appurtenances thereunto, both above and below ground, buildings, machinery and all tools, stores, supplies and merchandise on hand and all assignable outstanding contracts."

The city appointed as its appraiser Col. Henry A. Allen, consulting engineer of the Department of Public Works; the company appointed Mr. Harold Almert, of H. M. Byllesby & Company, Chicago. These two selected Mr. Leonard Metcalf, of Metcalf & Eddy, of Boston, as the third member of the board. These three engineers agreed on a few points, such as upon an original cost of the land and plant as $386,035 and a gross reproduction cost of the physical property as $296,370. But no appraisal of the property was possible because Colonel Allen differed radically from the remaining members of the board as to the value of the business as a "going concern" under the terms of the agreement, the depreciation of the physical property, and the market value of the land and buildings. He appraised the property at $208,582, while Messrs. Metcalf and Almert agreed upon $475,000 as a fair value under the terms of the appraisal agreement. As all three appraisers must agree upon a fair price in order to make such an estimate binding upon the parties to an agreement of this nature, the city decided to give up the reduction process, at least temporarily, and go ahead with an archaic method.

The appraisal of the going value of a business like that of the reduction company is manifestly an unusual event, but it is rendered quite simple when, as in this case, both parties agree in writing to their respective positions of willing seller and buyer. Under this condition the utility of the plant is manifestly assumed to be as great to the buyer as to the seller. The plant may need repairs, but its value as a going concern is self-evident when the alternative to its use is the filling of a deep excavation in a populous section of the city with putrescible material. In the face of such an alternative Colonel Allen's argument about going value in his report of Sept. 29 is difficult to understand.

In Colonel Allen's report he defends his position in respect to the appraisal on several grounds. In the first place, he states that it is practicable to handle the garbage immediately in a sanitary and inoffensive manner without taking it to the disposal plant of the company. This the Engineering Record doubts. Colonel Allen stated, in the second place, that the company had not operated its plant in a sanitary manner and its buildings were unsuited for garbage disposal by the best reduction methods, so that great depreciation and obsolescence items should be deducted from the first cost in order to reach the present value, even as a going concern. Mr. Metcalf, while of the opinion that the plant was not operated in conformity with present hygienic standards, contended that the expense of making necessary changes in it might reasonably be considered the measure of its functional depreciation, because the fixed and operating charges on this expense would probably have to be met from the sale of by-products and not from any larger payments by the city. In the third place, Colonel Allen held that the land of the company was worth about $112,500, while Mr. Metcalf held that it was worth about $116,200 plus about $15,000 for its special suitability as a site for garbage disposal, something which Colonel Allen refused to consider as existent. In the fourth place, Colonel Allen held that the plant was devoid of going value because of the expiration of the contract by which the city agreed to turn over its garbage to the company and to pay to it $47,500 a year for disposal. Mr. Metcalf pointed out that the city could operate the plant at a profit until it built its own station from more modern plans, which clearly showed that it had a going value as a business.

It is patent, of course, that this discussion is aside from the fact that the city was at perfect liberty to refuse to enter into any sale and purchase agreement in the first place if it believed that a new plant could be built at a saving over the purchase price of the company's plant. But having once agreed to buy and with the intention, too, of operating the plant, it is difficult to see how any interpretation other than the customary one, accepted by Mr. Metcalf, can be put on the term "going value."

The great discrepancy between the valuation by Colonel Allen, $208,582, and that by Messrs. Metcalf and Almert, $475,000, has caused discussion among engineers interested in such work. There have been cases before this appeared where appraisers did not agree, but no such conspicuous case where all the facts were so clear and where the conditions governing the appraisers were so lucidly set forth. For this reason the "Journal" of the proceedings of the Chicago City Council of Oct. 1, which contains the most important documents in the case, is likely to remain a valued part of any collection of papers treating on valuation work.

Design of Imhoff Sewage Plants—Part I

Some Fundamental Considerations and Suggestions Relating to Selection of Preliminary Screens and Proportioning of Grit Chambers

By Leslie C. Frank, Formerly of the Emscher River Board, Now Assistant Engineer, Baltimore Sewerage Commission, and Franz Fries, Engineer in Charge of Design and Construction, Emscher Board

The methods of design of Imhoff sewage tanks presented in this article are based upon seven years of practical experience in the Emscher District of Germany. For the conditions that exist in this district these methods have undoubtedly proved satisfactory. Twenty-two Imhoff plants, consisting of one hundred and twenty tanks, have already been constructed in the district under varying conditions and four more plants with thirty tanks are to be constructed this year. There are now in Germany about one hundred plants using this system.

The data herein given should not be used indiscriminately. Each individual case must be carefully studied beforehand and the design so modified as to fit that case. This can be done intelligently in most cases only in the light of expert experience.

Fundamental Data

First to be considered in the design of any disposal plant is the question of whether the sewerage system is upon the combined or upon the separate plan. If it is upon the combined plan more sludge will come to the plant than under the separate system, and this sludge will contain an appreciable quantity of detritus. It is necessary, therefore, to take due account of this increased sludge and of the detritus in designing the size of the sludge chamber and in determining whether or not a grit chamber should be placed before the tanks.

Important, too, is the matter of storm flows. Usually where Imhoff tanks are installed under the combined system a storm-water overflow is provided and so arranged that a given number of times the dry-weather flow is permitted to enter the plant, while the remaining excess overflows directly into the stream, or, as has been found desirable in some cases, it is caused to enter a storm-water basin. This basin has, for example, in the case of Essen Nord, a detention period of twenty minutes for storm flows of six times the dry-weather flow, at which time half of this sixfold flow enters the Imhoff tanks and the other half the storm-water basin. In the exceptional cases where the storm flow is even greater than sixfold dry-weather flow the dilution is considered sufficiently great to permit the excess to flow directly into the stream. After the storm the sludge from the storm-water basin is pumped back into the influent channel, taking the usual course through the settling tanks. In the method adopted at Essen Nord it is always necessary to pump the retained sludge from the storm-water basin into the influent channel. For cases where the fall at the plant is sufficient a special form of storm-water basin has been designed by Mannes which permits the retained sludge to be sent to the settling basins automatically. This has been carried out at Frohnhausen.

Where the separate system is in use the amount of sludge received is considerably less and the amount of detritus is reduced to a minimum. Hence under this system the sludge-room capacity is lessened correspondingly and a grit chamber is generally omitted.

The next factor that is considered in the design of the plant is the amount and character of the sewage. The character of the sewage affects greatly the time necessary for optimum sedimentation and the amount of this sedimentation. If the sewage is very dilute and contains no coagulating chemicals the detention period is made longer than if the sewage is very concentrated and contains such chemicals as iron wastes. Again, the production of hydrates due to the presence of chemical wastes generally increases the amount of sludge sufficiently to affect the size of the sludge chamber. The dilute sewages in the United States generally make necessary a much larger settling chamber compared with the sludge chamber than is the case in Germany.

It is the practice in Germany to determine the amount of sludge expected in a given plant upon the basis of population rather than upon the basis of volumes of sewage, and this basis of estimation has been found in most cases to be much the more accurate of the two. The population figure used in the design is generally that expected in five years. A design projected further into the future than this is not considered advisable, because one of the fundamental principles of Imhoff-tank operation is the necessity of having uninterrupted flow, and it is obviously uneconomical to build additional units and permit them to lie idle until the population of the community overtakes the size of the plant. Moreover, future advances in the science of sewage disposal may be more easily incorporated in the plant if the construction of units for future use be left, so far as economically possible, to the future.

The character of the population in a given community is unimportant in the design of disposal plants except insofar as it affects the amount of water used, and hence the volume of sewage.

It is important in any construction work to study the local conditions. If the ground material is of such a character that the ground-water table may be easily lowered, then the plant may be so designed as to permit of open-pit construction. If, however, the material is quicksand, or other conditions make the lowering of the ground-water table during construction difficult, then it may be advisable to sink the tanks as caissons.

Deep vs. Shallow Tanks

In the Emscher district deep tanks are preferred to shallow ones on the assumption that the composition of the sludge is thereby rendered better. Most of the tanks are made from 25 to 30 ft. deep. Where, however, the character of the ground material is such as to render excavation abnormally expensive it may be advisable to reduce the depth of the tanks somewhat.

It is the opinion of the authors that it is not necessarily true that shallower tanks mean poorer sludge, though this is probably true with tanks as at present designed. It is our belief that among the factors upon which depend the rapidity and thoroughness of decomposition in the sludge chamber are intensity of mixing action due to gas ebullition and thoroughness with which the incoming particles of fresh sludge are distributed over the cross-section of sludge room. Since each cubic foot of properly decomposing sludge gives a certain intensity of ebullition, it is obvious that the deeper the tank the more intense is the ebullition. Therefore, the deeper the tank the greater is the rapidity of decomposition.

It would be logical to conclude, then, that in order to attain the same degree of decomposition with shallower tanks it would simply be necessary to increase the time of decomposition. In order to increase the time of decomposition, however, the sludge room must be made larger. Hence the tendency will be to expand it laterally, and this, with present designs, leads to a greater cross-section area of sludge room per unit of slot feeding fresh sludge—in other words, the thoroughness with which the incoming particles of fresh sludge are distributed over the cross-section of sludge room is lessened. Whether this is sufficiently important materially to affect the rapidity of decomposition can only be shown by future experience. We believe, however, that engineers in the past have failed to recognize the fact that shallow tanks need a longer period of decomposition than deep tanks, and that this is the reason why shallow tanks in the past have yielded a poorer sludge. Whether it will be possible to design the tanks so as to meet this difficulty is uncertain, as the necessity of making the sludge chamber larger would seem to militate against keeping it shallow.

The fall necessary in Imhoff plants between the influent and effluent channels may be placed as low as 1 to 2 ft. in cases where grit chambers are not used. Where grit chambers are used, however, and it is desired to drain the retained grit by gravity, a total fall of from 3 to 4 ft. in the plant is necessary. To avoid this additional fall, however, the grit drainage water may be pumped out.

Preliminary Screens

The object of the preliminary screens is to prevent the larger solid substances from getting into the sludge room and clogging the sludge pipe. The clearance used in these coarse screens is usually 2 to 2½ in. The reason for such large clearance is to prevent the retention of fæcal matters which should logically enter the tanks. In the older practice it was usual to hold back a large proportion of the solid fæcal matters at the screens, and in the Emscher District this practice has been abandoned as illogical. Due to this relatively large clearance, therefore, the volume of material held back by the screens is small.

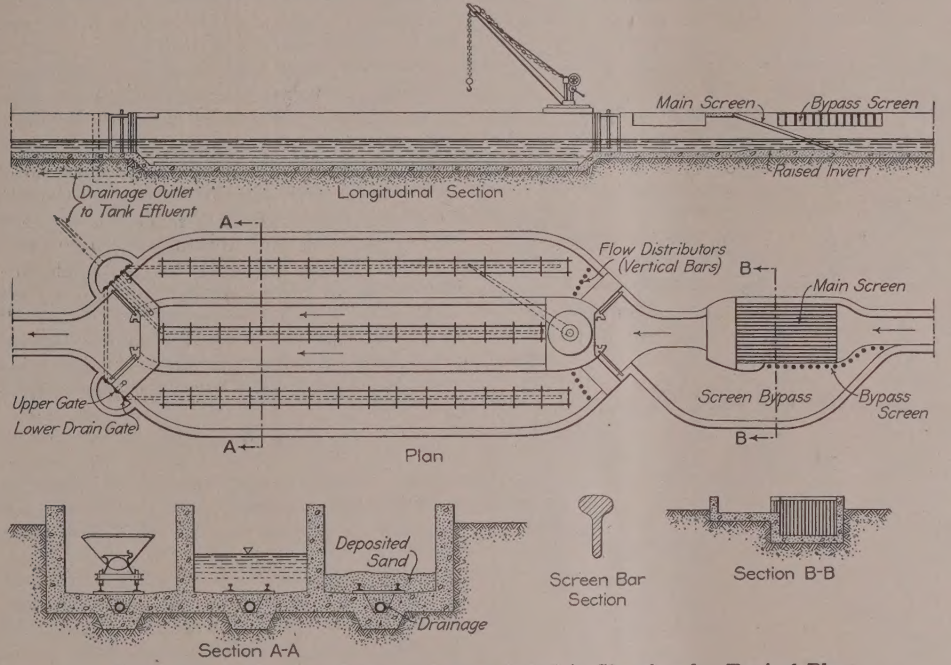
The total width of the screen chamber is usually made 1½ to 2 times the width of the influent channel, so as to minimize friction and consequent damming. Therefore it is considered advisable, especially in large plants, in order to maintain the necessary velocity to prevent sedimentation, either to raise the invert under the screens, so as to reduce the cross-section to that in the influent channel, or to increase the slope under the invert.

The inclination of the screens is about 1 vertical to 3 or 4 horizontal. This is a greater inclination than has been the practice elsewhere; but this greater inclination has many advantages. First of all, it insures a lessened encroachment upon the fluid cross-section, since the more the bars are inclined toward the horizontal the less is their vertical cross-section. Again, the useful superficial area of the screens is increased by this greater inclination. Then again, the flow of sewage tends to shove the entrained matter toward the upper part of the screen and thus minimizes clogging. Finally, the cleaning of the screen is facilitated by the greater inclination.

The cross-section of the bars depends upon the necessity for stability and upon the need of minimizing clogging. In this district the cross-section shown in the drawing has been much used, as it provides a maximum stability, due to the bulbous head, and insures minimum clogging, due to the narrow web.

The lateral brace bars are generally placed at the invert end of the screen, at a point somewhat above the water surface, and at the upper end of the screen. None should be placed within the fluid cross-section, as then the entrained matters are not easily removed.

A screen by-pass is usually provided in the larger plants of the Emscher district, as shown in the drawing. Its purpose is to prevent flooding of the plant if the screen proper should accidentally become superclogged, which may happen during a rainstorm at night, when there may be no attendants at the plant.



Details of Preliminary Screens, Bypass and Grit Chamber for Typical Plant

The screen is simply hand cleaned as often as necessary with a suitable rake.

GRIT CHAMBER

The object of the grit chamber is to retain the harmless mineral suspended content—chiefly street grit—of the sewage. Such suspended mineral matter would merely lessen the capacity of the sludge chamber if it is not previously removed from the sewage, and in addition it is liable, when arriving in considerable volumes, to result in stoppage of the sludge-discharge system. Such harmful volumes of grit do not generally appear where the separate system of sewerage is in use, as has been previously mentioned, and therefore in this case the grit chamber is usually omitted. Neither are grit chambers included in the case of small plants on the combined system where the conditions are such as to insure small amounts of grit.

In the sewer the heavier sand particles are being rolled along the bottom and the lighter ones repeatedly thrown back into the lower part of the fluid cross-section. In the grit chamber the velocity of flow should be so reduced so that these lighter particles are no longer thrown back, but tend to roll along the bottom or remain quiet. Such an average velocity has been found by experience to be about 1 ft. per second.

The depth of the grit chamber is limited by the necessity of preventing the deposit of organic sludge. From this standpoint the difference between a grit chamber and a sedimentation chamber in theory may be regarded to be the fact that in the sedimentation chamber there is provided a lower "slow-motion zone" in which the settling organic sludge particles may be entrapped, while in the grit chamber this slow-motion zone is simply omitted so as to prevent such entrapping. Therefore a properly constructed grit chamber should be essentially shallow. In this district the practice is to place the invert of the grit chamber 6 in. to 1 ft. below the sewer invert. Another distinct advantage of these shallow grit chambers is that the absence of a lower slow-motion zone and the consequent fairly rapid passage of the sewage over the surface of the entrained grit tend to wash the grit practically free of organic matter. Experience in this district has shown that such properly designed grit chambers actually retain only clean grit or sand, which may be used as such without fear of putrefaction. It is often used in the Emscher District for surfacing the paths of the plants.

The width of the grit chamber depends, of course, upon the above-determined depth, the above-mentioned velocity, and the volume of sewage flow.

The length of the chamber should be such as to insure the deposit of an optimum per cent of the sand and to provide sufficient sand-storage volume to avoid the necessity for too frequent cleaning. Such a length in practice has been found in the Emscher District to be about 70 to 100 ft., as this gives an ample detention period of from 60 to 90 seconds, and in addition provides the required storage.

The invert of each grit chamber unit is usually provided with a longitudinally placed drainage pipe, closed at the outlet end by a valve. This drainage pipe is placed in a depression in the grit chamber invert and surrounded, up to the level of the invert, with graded slag or gravel, the top layer being sand. The pipe is usually ordinary agricultural drain tile, placed loosely end to end. The grit chamber is composed of several units (generally three). The extra units are provided for two main reasons—(1) to permit taking a unit out of operation while it is being cleaned and still retain the grit chamber function; (2) so that during storm flows more than one unit may be taken in operation. Otherwise the velocity of flow in that one unit would be so increased as to flush the sand it had previously retained into the tanks. In the past where grit chambers have been designed for maximum storm flow it was found that putrescible sludge settled out with the grit during ordinary flows, and where grit chambers were designed for ordinary flows maximum storm flows flushed out the sand. It is advisable, therefore, to provide a number of units, each one designed for ordinary flows.

In the invert of the larger grit chambers are usually placed tracks for hand cars. In emptying the grit chamber the hand cars are loaded with grit and then shoved to the end of the chamber, where the container is lifted out by means of a hand crane.

This is the first of a series of three articles on Imhoff plants by Mr. Frank and Mr. Fries. The second article, to appear in next week's issue, will discuss the design of the settling-chamber compartment of the tanks.

English Test for Dissolved Oxygen

By George C. Whipple, Professor of Sanitary Engineering, Harvard University, Cambridge, Mass.

The amount of dissolved oxygen that will be absorbed from fully aërated water by the organic matter contained in sewage or a sewage effluent in a definite time at a prescribed temperature is a test much used in England and has been adopted as one of the standard tests by the Royal Commission on Sewage Disposal. These standards were set forth in the commission's eighth annual report issued a few months ago and described in the Engineering Record of Dec. 28, 1912, page 711. The details of the method were not given in this report, but they have subsequently appeared in an appendix published as a second volume of the report. On account of the interest which this subject has for American sanitary engineers the lengthy description in the report has been condensed below.

A measured volume of the sewage or sewage effluent to be tested is thoroughly shaken, so as to obtain as complete a reoxygenation as possible, and mixed in a flask with a larger measured volume of tap water, and again shaken gently, but thoroughly. The suspended solids should not be removed before making the test. The dilutions required for raw sewages and the effluents of settling tanks are about 99 to 1 and 49 to 1 respectively, and for sewage effluents about 4 to 1. The intention is to so adjust the ratio of aërated water to the liquid being tested that during the test only about 50 or 60 per cent of the dissolved oxygen in the diluting water will be used up. It has been found that if less than 30 per cent of the supplied oxygen is absorbed the error of the experiment becomes large, but that if more than 60 per cent is absorbed the error is even greater. Too great a dilution reduces the food supply of the bacteria so that absorption of oxygen is retarded.

The method is carried out as follows: Four scrupulously clean, glass-stoppered bottles holding from 340 to 360 cu. cm. are filled to overflowing with the diluted sample and left unstoppered for five minutes. Any bubbles of air that tend to lodge at the shoulder must be dislodged, after which the glass stoppers are carefully inserted so as to entrap no air. It is important that the sewage or effluent to be tested, the water used for dilution and even the bottles themselves be at substantially the temperature at which the mixture is to be incubated, namely, at 65 deg. Fahr. Otherwise errors will arise due to the change in solubility of oxygen at different temperatures.

Two of the bottles are tested for dissolved oxygen immediately. The other two are put in a 65-deg. incubator and kept for five days, and then tested for dissolved oxygen.

The temperature of 65 deg. Fahr. (18.3 deg. C.) was selected because in England this is practically the monthly maximum. It is said that the summer temperatures of even the most sluggish streams seldom exceed it unless trade wastes are allowed to flow in directly.

The five-day period of incubation was chosen partly because it has been much used in England and partly because it has been found to yield results which are well correlated with the character of similar mixtures of sewage and water in natural streams, as determined by long-continued observation, but also because the error of experiment has been found to be less than when a shorter period of incubation has been used.

Laboratory Routine

The method of determining the amount of dissolved oxygen present is that of Winkler as modified by Rideal and Stewart. To the mixture in the bottle is added 0.9 cu. cm. of concentrated H_2SO_4 and then sufficient N/8 permanganate solution (3.94 grams $KMnO_4$ per liter) to leave a slight excess of the latter after the whole has been mixed and has stood, stoppered, for twenty minutes. Usually 1 or 2 cu. cm. are sufficient. After this necessary period of twenty minutes has elapsed the excess of permanganate is destroyed by adding about 1 cu. cm. of potassium oxalate (2 per cent of the crystallized salt, $K_2C_2O_4 + H_2O$), the bottle being at once restoppered and its contents mixed. When the liquid has become colorless, the stopper is again removed and 1 cu. cm. of manganous chloride solution (33 per cent of the crystallized salt, $MnCl_2 + 4H_2O$) run in from a pipette at the bottom of the bottle, and immediately afterward 4 cu. cm. of the mixed potash and iodide solution are added (70 grams KOH and 10 grams KI per 100 cu. cm. of distilled water). The stopper having again been inserted, the bottle is inverted once or twice to mix the contents and then again after standing a few minutes. The second rotation insures a good settlement of the hydroxide of manganese.

Five cubic centimeters of concentrated HCL (free from chlorine) are run in from a pipette and the bottle restoppered and kept in the shade for five to ten minutes, with occasional rotation. Then 20 cu. cm. of the liquid are pipetted out of the bottle and rejected and the remainder is titrated with N/20 sodium thiosulphate (12 grams per liter), using some clear starch solution toward the end of the titration. For practical purposes no correction need be made for the 8 cu. cm. or so of added reagents, the maximum error of dissolved oxygen absorbed being less than 2 per cent.

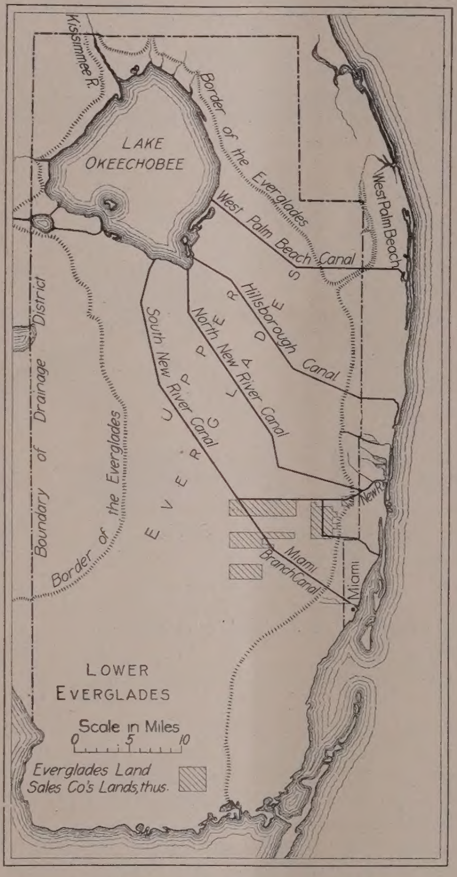
The calculation is made as follows: If N is the number of cubic centimeters of thiosulphate used, S, the strength of the thiosulphate expressed in grams of oxygen per cubic centimeter, V, the volume of the bottle in cubic centimeters, then the dissolved oxygen in parts per million will be $1{,}000{,}000\,NS/V$. To ascertain the amount of dissolved oxygen absorbed, the difference between the amount present before and after incubation is found and multiplied by the dilution of the mixture.

The English standard for sewage effluents is one that absorbs less than 20 parts per million of oxygen in five days at 65 deg. Fahr., the test being made as indicated above.

Reclaiming the Florida Everglades

Joint Report of Board of Engineers, Daniel W. Mead, Leonard Metcalf and Allen Hazen, States Drainage Project Is Technically and Financially Feasible

The drainage of the Everglades in southern Florida has been the subject of so many personal disputes in and out of Congress that comparatively little attention has been paid of late to the engineering problems of the undertaking, except by the Everglades Land Sales Company, of Miami. In July, 1912, that company retained Messrs. Daniel W. Mead, Leonard Metcalf and Allen Hazen as a board of engineers to ascertain whether the main canals which the State was constructing for the drainage of the Upper Everglades would accomplish their purpose, and, if not, what amplification of those works will be needed.



Map of the Everglades

The results of the investigation were summed up by the engineers as follows:

Summary of Findings

"We find the drainage of the Florida Everglades feasible both from an engineering and financial point of view, but we are of the opinion the development must be a gradual one.

"We find the present and projected system of canals, as provided for by the State of Florida, totally inadequate to accomplish the drainage of the Everglades.

"We are of the opinion that the reclamation of your lands can best be accomplished by diking, ditching and the construction of pumping stations by which the water may be drained from your lands."

These findings, so greatly at variance with previous published statements on the subject, have been followed by the active prosecution of work by the company to carry out the recommendations of the engineers, so that the 70,000 acres of Everglades land which it either owns or has already sold may be made fit for agricultural purposes.

Southern Florida is underlaid by a bed of coralline limestone. There is a ridge in this limestone, 3 to 5 miles wide, between Lake Okeechobee and the Atlantic, and running southerly with gradually decreasing elevation until it disappears south of Miami. There is another ridge, somewhat wider, with its backbone 20 miles west of Lake Okeechobee, which extends southward, parallel with the first. In this way a great tray of limestone is formed having Lake Okeechobee at the north and the Everglades running south from it to the end of the peninsula.

The Everglades have an area of about 4000 sq. miles, according to a report by Lieut. J. C. Ives. The limestone is overlaid by sand, alluvial matter and decayed vegetation, forming a mass of muck from 3 to 10 ft. or more deep. On the muck is a sheet of water seldom deeper than 3 ft. during the dry season. The elevation of the water fluctuates from 2 to 3 ft. There is a heavy growth of saw grass, except on the small islands, which are always dry; here semi-tropical vegetation is found. The general southerly slope of the Everglades from Lake Okeechobee is about 3 in. per mile; west of Miami the surface is about 8 ft. above sea level.

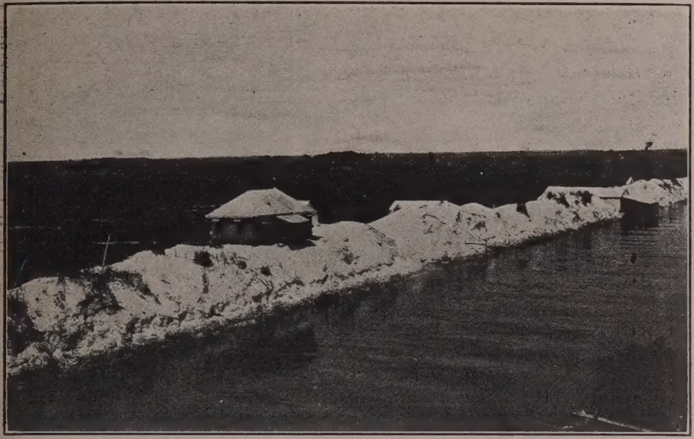
Lake Okeechobee

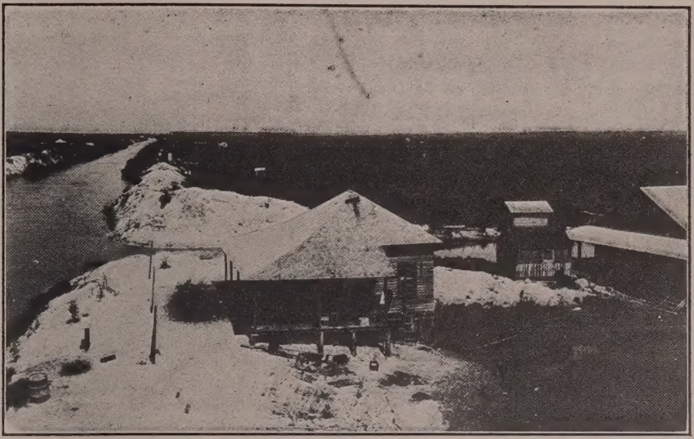
Lake Okeechobee has an area of about 733 sq. miles and a drainage basin of about 5366 sq. miles, according to Capt. H. H. Slattery. Maj. J. O. Wright has given 710 and 6077 sq. miles for these areas. Formerly it had no clearly defined outlet to the sea, and the water slowly drained away through the Everglades. In 1884 a canal was cut from the lake to the Caloosahatchee River, and since then this channel has afforded some drainage to the Gulf of Mexico. During very high water in the river, however, there is a backflow through the canal into the lake. On the south side of the lake are numerous small channels through which water flows during high water into the Everglades, where their course is eventually obliterated in the expanse of saw grass. During low water in the lake the flow in these channels is in the opposite direction. The elevation of the lake fluctuates between about 19 and 22 ft. above sea level, the full stage being at about 20.6 ft.

On the north the lake is fed by the Kissimmee River, which overflows its banks during floods and covers a strip of country from 1½ to 2 miles wide. The whole drainage area is flat and covered with vegetation.

The Caloosahatchee River, into which the canal from the lake normally discharges, is a sluggish, meandering stream from 6 to 12 ft. deep. The board of engineers did not consider it an important factor in the drainage of the lake, because its fall and cross-section are inadequate to carry even its own storm waters without flooding its banks.

The control of Lake Okeechobee has been considered by all engineers who have reported on the reclamation of the Everglades to be essential for the success of the work. Inundations of the Everglades from the lake must be prevented before the drainage of the land can be undertaken profitably, while the lake will be needed to store water for dry-weather irrigation of the land after it is drained. The latter use will eventually become the more





View of Everglades Canal at "Zona," the Community of Retired Panama Canal Workers

important, in the opinion of the board of engineers.

The bottom of the lake slopes gently to maximum depths of about 20 ft., more or less. A minimum depth of 16 ft. has been established for navigation purposes by the Corps of Engineers, U. S. A. The storage capacity of the lake between El. 16 and 19 is approximately 1,363,000 acre-ft.

State Drainage Work

The drainage of the Everglades is under the trustees of the Internal Improvement Fund of Florida, a body created by the Legislature in 1855. For about two years prior to August, 1912, the work was under the direction of Maj. J. O. Wright as chief engineer. In 1907-8 he investigated the reclamation of the Everglades for the U. S. Department of Agriculture, and afterward was engaged by the State to carry it out.

In his original investigation he decided that to prevent floods in the Everglades it would be necessary to remove 3938 sec.-ft. by means of canals during the rainy season. Later he concluded that the removal of 3200 sec.-ft. would be ample to control the lake, except possibly in extraordinary storm periods occurring at rare intervals. His plans provided for an outlet into the Caloosahatchee River, now completed, except for additional dredging ordered by the Government, four canals now under construction, one of which was not fully planned in August, 1912, and two that had not been studied in any detail at that time. It has been the purpose of the trustees to cut other canals as they became necessary and funds were available for the work.

The board of engineers makes the following statement regarding the official announcements concerning the canal: "The State officials have not apparently given formal notice that these works would drain the Everglades, but it appears that the trustees of the Internal Improvement Fund have been led to hope that they might be sufficient, and the public generally has quite definitely assumed this to be the case. It is hardly conceivable that the individual owners of small tracts of land in the Everglades, who probably number from 20,000 to 30,000, would have bought their holdings had they not presumed that the works now under construction by the State would accomplish the reclamation of the Everglades and make their lands tillable."

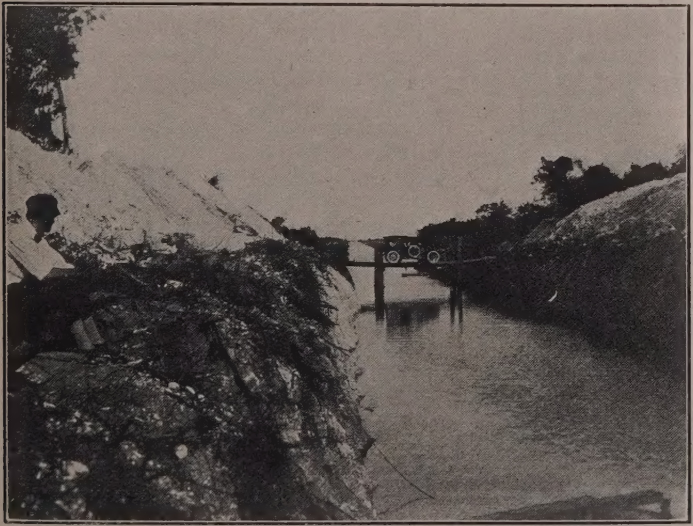
The board of engineers further reports that the capacity of the State's canals, instead of being 3200 sec.-ft., will not be more than about 2800 sec.-ft. under the most favorable conditions. The State's canals are not low enough to afford adequate drainage, irrespective of their width.

Basis of State Plans

The determination of the dimensions of the canals undertaken by the trustees has been explained by Major Wright substantially as follows: He assumed that the rainfall records at Kissimmee, in the northern part of the drainage basin of Lake Okeechobee, were representative of the entire watershed. At this place the maximum annual precipitation during a nine-year period was 70.92 in., the minimum was 40.22 in., and the average was 53 in. This annual rainfall is not uniformly distributed but is much greater in the summer and fall. There was a record of 27.95 in. in July and August, 1905, but this was considered a locally heavy precipitation at Kissimmee, and the rainfall on the entire watershed at that time was believed to be not more than 26 in. Rainfalls of from 18 to 22 in. in two consecutive months had been recorded three times in the period under consideration, and consequently it was assumed that the maximum on the whole area for two consecutive months was not likely to exceed 24 in.

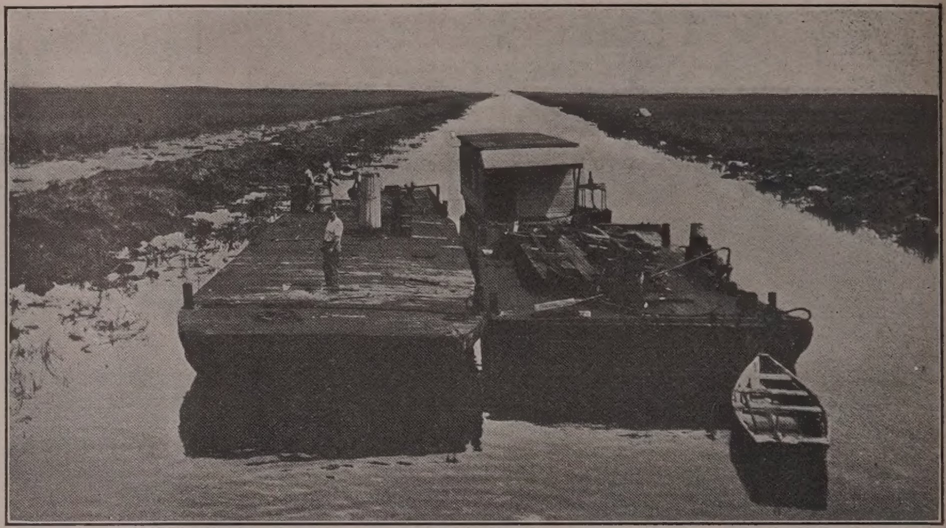
It was assumed from a study of evaporation investigations made in several places in the United States and Europe that the evaporation during such conditions as obtained in July and August, 1905, would be 75 per cent of the rainfall. The assumed precipitation for designing purposes was 24 in. in sixty-two days, or 0.387 in. daily. Subtracting 0.3 in. from this to allow for evaporation gave 0.087 in. a day as the mean daily runoff from the entire watershed for which provision must be made.

The land surface discharging into the lake being about 7½ times the area of the lake, and the runoff being assumed as 0.087 in. daily, the lake would be raised 7½ times 0.087 in., or 0.65 in. daily, it was figured. This would be equal to 40.45 in. during the two months. The daily evaporation from the lake being estimated at 0.25 in. and the rainfall at 0.387 in., there would remain 0.137 in. daily, or 8.49 in. in two months, to be added to the runoff from the land surface. The total thus reached was 48.94 in. as the elevation of the lake that might be occasionally expected during July and August, could the banks be raised to store all the storm water.



Ditch Showing at Left Rock-Rim Boundary of Everglades

Movable Survey Camp Mounted on Caterpillar Tractor



Main Canal in the Everglades, Where No Rock Was Encountered

From these assumptions it was considered desirable to dig canals from the lake to tidewater of sufficient capacity to reduce its elevation to 16 ft. above sea level just before the beginning of the rainy season, and allow a storage capacity for 36 in. of the 48.94 in. of total runoff liable to reach the lake. On this basis the canals would have to remove 18.94 in. in 62 days, or 3938 sec.-ft. The canal system was estimated to have a capacity of 3993 sec.-ft. when fully completed and in perfect condition.

The Elliott Report

These recommendations were reviewed by Mr. C. G. Elliott while chief of the Drainage Bureau of the Department of Agriculture, who at first gave them perfunctory endorsement, but subsequently concluded that the proposed capacity of the canals was inadequate and made certain changes in the basic figures in Major Wright's report. He was of the opinion that "examinations have not been made in sufficient detail to secure the facts that are necessary to an intelligent design of a drainage plan for the Everglades." The area to be reclaimed and the conditions on it were regarded by him as without counterpart in any completed project. His suggestions were put forward, therefore, as indicating the lines which should be followed when more reliable information regarding local conditions was obtained.

He accepted the Kissimmee rainfall records as representative of the whole drainage basin and assumed a maximum precipitation of 24 in. in sixty-two consecutive days, as did Major Wright. He assumed a somewhat smaller amount of runoff, however—22 per cent—which would give 5.28 in. of runoff in two months. This would raise the lake surface 39.6 in., by applying the same reasoning as before. The rainfall on the lake less the evaporation would be 5.28 in., giving a total rise of 44.88 in., should no discharge occur in that time. If it is assumed, as before, that the lake is drawn down to El. 16 at the beginning of the wet season, and it is allowed to rise 36 in., 8.88 in. remain to be removed by canals. Taking into account such differences as exist between areas which have been successfully drained and the Everglades, Mr. Elliott recommended designing the main ditches of sufficient capacity to remove ¼ in. in depth of water in twenty-four hours, which is equivalent to 6.72 cu. ft. per square mile. He stated, moreover, that there was an uncertainty in this matter which could only be removed by constructing a series of drains which would be experimental to some extent. The canals he suggested had a total capacity at the lake of 2998 sec.-ft.

The board of engineers states that, "cautious as the Elliott report is, we are of the opinion that it is still too optimistic, and that the capacities suggested therein are not so great as the available data indicate to be necessary for the final success of the work."

Rainfall and Runoff

Records of the U. S. Weather Bureau show that the rainfall on the Everglades is from 36 to 65 in. annually, averaging 53 in. This average is about that of Louisiana, upon which reliance was largely placed in early studies of Everglades drainage; but its distribution is not so uniform, so that the number of months of rainfall exceeding 10 in. is very much greater in Florida than in Louisiana. Individual storms of 2½ in. or more daily are more frequent in Louisiana than in southern Florida, but the Florida storms of this nature frequently last for longer periods and result in more adverse drainage conditions.

In order to study the distribution of the rainfall throughout the Everglades the board of engineers platted the daily rainfall at the Florida observation stations and also the total rainfall for the three months of maximum rainfall for each year from 1891 to 1911. By drawing lines of equal rainfall on these maps it is believed that a better idea of the actual distribution of precipitation can be obtained than in any other way. The intensity of the rainfalls which are revealed in this way leads the board to believe that the runoff in the Everglades will be greater than in Louisiana, where provision is usually made for removing ¾ to 1 in. in twenty-four hours from areas of 10 sq. miles or more.

Approximate records of a flood in the Caloosahatchee River in June, 1912, indicate that about 12,000 sec.-ft. were discharged from an area of about 500 sq. miles, equivalent to 0.9 in. per twenty-four hours. During the same storm Lake Okeechobee rose 12 in. in seven days and continued to rise less rapidly for some time subsequently. This indicated an average discharge of 0.2 in. per day over the whole area for a seven-day period, with the probability that the maximum discharge was as much as 1¼ in. in twenty-four hours.

From all the information it could obtain the board of engineers concluded that for complete drainage, when the areas to be reclaimed are fairly concentrated, the canal capacities should provide for a discharge of about 1½ in. per twenty-four hours, or 40 sec.-ft. per square mile of drainage area on land tracts of 50 sq. miles or less; not less than ¾ in. per twenty-four hours, or 21 sec.-ft. per square mile of tracts of about 200 sq. miles; not less than ½ in. per twenty-four hours or 14 sec.-ft. per square mile or 14 sec.-ft. per square mile on tracts of about 750 sq. miles, and not less than ¼ in. per twenty-four hours, or 7 sec.-ft. per square mile of the entire Everglades if the discharge were concentrated in a single canal.

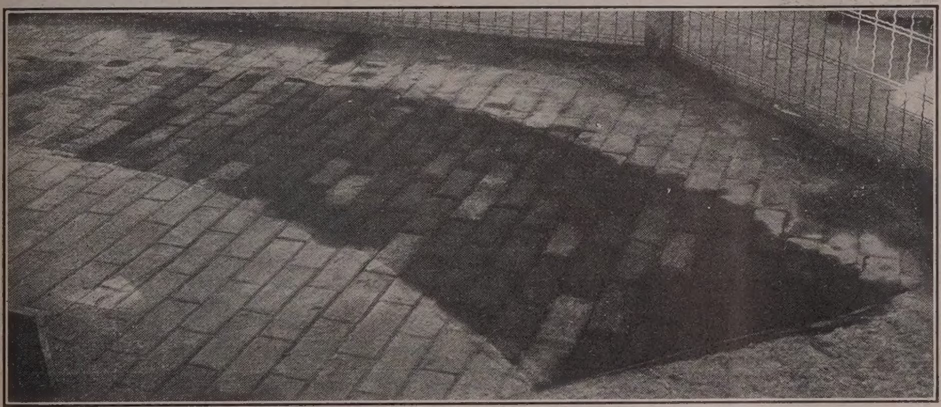
The board did not believe that an investment to avoid floods that occur only at long intervals was warranted. Its intention was only to indicate adequate provisions for floods likely to recur with sufficient frequency to render it advisable to provide for their complete removal. The board also believed that eventually the water stored in Lake Okeechobee would prove so valuable for irrigation that it will become advisable to increase its storage capacity by constructing dikes at the low points along its margin.

In the early stages of the reclamation, while the water is not required for irrigation, the board believed that it would be possible to hold the lake level ordinarily at a low stage and to use the lake storage temporarily for flood waters. Under these conditions half the flood flow may be stored temporarily and canals having a capacity of not more than ⅛ in. runoff in twenty-four hours may be adequate. This would require a canal with a capacity of 20,000 sec.-ft., say 500 ft. wide and 15 ft. deep from head to outlet. For the discharge of the full flood occurring when the Everglades are completely developed the capacity of the canal must be 40,000 sec.-ft. The drainage of the lake should be through canals intended for that purpose exclusively, and not for the drainage of the Everglades.

The board of engineers reported that the proper reclamation of the Everglades would

Machine for Cutting Drainage Ditches in the Florida Everglades be a very slow process. Even under existing conditions there is a great scarcity of transportation facilities to handle the produce of the region along the east margin of the district. The work must progress no faster than the land can be utilized, because it grows up to jungle if not cultivated as soon as drained, and the expense of clearing the jungle is as great as that of drainage. "As the problem of supply and demand becomes progressively adjusted by development of transportation facilities, by rate adjustments, by a thorough knowledge of agricultural possibilities and practicabilities, as determined in the light of all these factors, there can be no doubt that the entire Everglades can and will be successfully reclaimed."

The board concluded that in no case which it saw could the owners of land within the Everglades expect effective drainage, so far as their individual lands were concerned, from the work projected by the State of Florida, nor could it be anticipated that the State would be able to give early relief from existing conditions. The board advised applying the State funds to a thorough study of the entire problem, to furnishing adequate outlets to the ocean, and to the control of Lake Okeechobee. They regarded the successful reclamation of the private lands within the Everglades as largely dependent on private effort, and they advised a system of diking and pumping for the lands of their client. The problem is complicated by the absence of any drainage act under which the work can be carried out and by the policy of the State in selling alternate rather than contiguous sections of land. Both of these defects must be remedied, the board reported, before the work can be properly carried on.



Section (After Test) of Brick Pavement Laid on Ordinary Sand Cushion

Tests of Concrete and Brick Pavements

Details of the Types Tested with the "Determinator" at the American Road Congress in Detroit, and a Statement of the Obvious Results

During the American Road Congress at Detroit tests were made on concrete and brick pavements by means of the "determinator," a machine designed for the Department of Public Works of Detroit. A space 40 ft. square, being part of the exhibit space of the Universal Portland Cement Company, was prepared for the test, and was divided as follows. In half of the space a 1:1½:3 gravel-concrete pavement was laid by the Wayne County Highway Commission; in one-fourth of the remaining space crushed granite concrete and in the remaining quarter a brick pavement, composed of two sections similar in all respects except that one had a 1:5 mortar cushion and the other an ordinary sand cushion. The machine was started at noon on Monday, Sept. 29, and was stopped finally at noon on Saturday, Oct. 4, making in that time 19,315 revolutions. The results are shown by the photographs and are summarized briefly at the end of this article and in the editorial comment in this issue, to which attention is particularly directed for an estimate of the value of the test.

Foundation

At the site of the exhibit at the corner of Second and Front streets the earth was excavated to a depth of several inches. A reinforced-concrete foundation slab was built over the whole area, and on this, as a subgrade, the pavements were constructed.

One-half of the area, a space 20 x 40 ft., was paved with concrete, built by a gang supplied by the Wayne County Highway Commission, using the materials, proportions, methods and tools utilized on the county highways. The coarse aggregate was screened, washed gravel, graded in size of pebbles from ¼ to 1½ in. This material is supplied to the Wayne County Highway Commission by Lennane Brothers, of Pleasant Lake, Ind. Its analysis will be found in Table 1.

The fine aggregate was washed sand, consisting of hard, silicious particles well graded in size from the finest to the largest particles, all passing a ¼-in. opening. This sand was also supplied by the Wayne County Highway Commission, and is also furnished by Lennane Brothers. The grading is given in Table 2.

Gravel and sand were measured in wheelbarrows after first instructing the laborers as to the amount of material for a measured load. The concrete proportions were 1 sack of cement, 1½ cu. ft. of sand and 3 cu. ft. of gravel. Universal Portland cement was used.

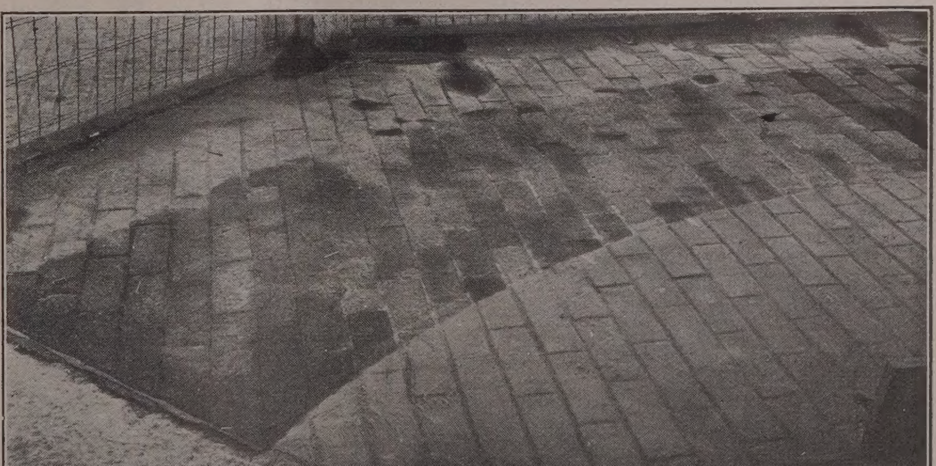
The concrete materials were mixed in an electrically driven ½-yd. batch mixer, and for each batch the mixing was continued for one minute after all materials, including water, had been placed in the mixer. This mixer turned 19 r.p.m. The concrete was mixed to such consistency that it would flow somewhat when deposited from the wheelbarrows. It could be shoveled about without running off the shovel to any extent, and after working down with the template and floating a very small amount of water appeared on the surface. The concrete was 7 in. thick.

Joint protection plates of the Baker type were placed against the forms which divided the concrete from the remaining part of the area to be paved, and a protected joint was made across the gravel-concrete pavement, dividing it into two sections, using the Baker plates.

As soon as deposited, the concrete was struck off with a heavy template resting on the side forms and moved back and forth across the surface, advancing a few inches forward at each move of the template. After compacting and molding to a truly plane surface in this manner the surface was finished with wooden floats from a bridge supported outside the edges of the concrete. The floating was done before the concrete had set up to the consistency common in sidewalk or floor construction, the purpose here being simply to remove irregularities in the surface and work down any coarse stones, leaving a gritty, rough surface. Finishing was completed in less than 2 hours after depositing concrete.

After hardening overnight the surface of the concrete was covered with sand and kept wet for a period of three weeks. The pavement was built on Aug. 17.

The concrete pavement built by R. D. Baker was laid Aug. 22. Crushed Wisconsin granite, supplied by the Wisconsin Granite Company, of Chicago, Ill., in sizes graded uniformly from ¼ to 1½ in. was used as coarse aggregate. (See Table 3.) Sharp sand, supplied by the C. H. Little Company, of Detroit, obtained from Fleming Creek, Ypsilanti,



Section (After Test) of Brick Pavement Laid on 1:5 Mortar Cushion

Mich., was the fine aggregate used. The proportions were 1 sack cement, 1½ cu. ft. sand and 3 cu. ft. crushed granite. The sand analysis is given in Table 4.

The concrete was mixed and placed as described for the gravel concrete. A protected joint was made diagonally across the slab, the plates being placed with the Baker patent installing bar. This pavement was 7 in. thick.

Brick Pavement

A standard vitrified brick pavement was laid in the remaining one-fourth of the exhibit space. Champion brick were supplied and laid by Mr. I. M. Adams, of the Detroit Vitrified Brick Company. In half of this space, or one-eighth of the circle, brick were laid on a 1:5 mortar bed. This was prepared by mixing the sand and Portland cement dry and spreading on the concrete base in the same manner as is the ordinary sand cushion. After the brick were laid water was applied from the surface and the mortar allowed to harden.

Table 1—Granulometric Analysis of Gravel Used in Test Pavement

Sieve	Per cent retained, by weight	Sieve	Per cent retained, by weight
1¾ in.	0	No. 2	37.1
1¼ in.	9.1	No. 4	25.8
¾ in.	18.9	Through 4	9.1

Per cent voids, 32.4.

Table 2—Granulometric Analysis of Sand Used in Gravel—Concrete Section of Test Pavement

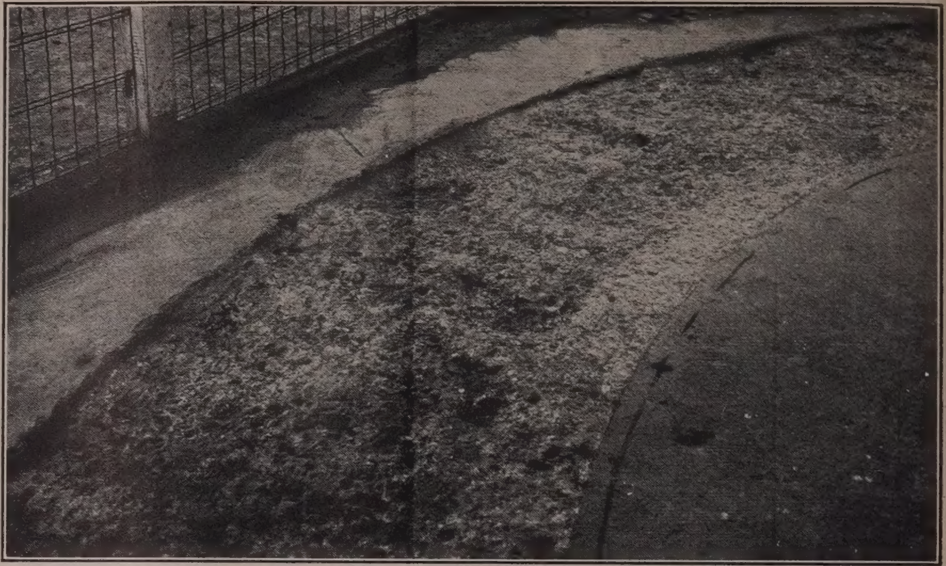
Sieve No.	Per cent retained, by weight	Sieve No.	Per cent retained, by weight
4	0.0	50	6.3
10	35.9	80	6.8
20	26.5	100	1.0
30	10.9	200	0.7
40	11.3	Through 200	0.6

Per cent silt, 2.30; per cent voids, 28.0; specific gravity, 2.65; strength of 1:3 briquettes (average of 3), at 3 days, 303 lb. per square inch, at 7 days, 358 lb.

The remainder of the brick were laid according to the specifications of the National Paving Brick Manufacturers' Association. A 1½-in. cushion of sand was spread over the concrete base to support the brick.

A 1:1 Portland cement grout was applied in the usual manner over both sections.

Champion block are 3½x4x9 in. in size, with lugs about ⅛ in. high on one side; corners are rounded to a 3/16-in radius. The brick are a first-class vitrified shale paver, made at Corunna, Mich. A standard rattler test on samples of brick selected from this lot showed average losses of 34.7, 29.9 and 31.0 per cent.



Section (After Test) of Concrete Pavement Made with Gravel Aggregate

The brick were carefully laid and a smooth surface was obtained without appreciable depressions or projections. A pounder was used in the ordinary manner and the surface was finished satisfactorily without the use of a roller. The surface was covered with sand and kept wet for a period of 7 days.

The paving determinator used in this test was designed for the Department of Public Works of Detroit by Mr. John C. McCabe, city boiler inspector.

The determinator consists of a central vertical column or pivot around which revolves a horizontal shaft having heavy cast-iron wheels on each end. The wheels are 48 in. in diameter, with rims 3 in. wide. The weight transmitted to the floor by each wheel is 1650 lb. While the wheels remain at a constant distance apart, as they move around the central pivot the radius of one increases and that of the other decreases, so that each wheel travels over a path measuring 27 in. wide from inside to outside of wheels. The greatest wheel radius of the machine is 10 ft. 10 in., the least radius of inside edge of wheel is 8 ft. 7 in., and it requires approximately 330 complete revolutions for a wheel to shift from the inside to the outside of the circular path.

To reproduce the wear due to rough-shod horses, each wheel is provided with five plungers, on the ends of which are metal plates shaped like the bottom of a horse's hoof, each plate having four steel calks. As the wheels revolve, these plates strike the surface of the pavement a heavy blow, regulated by means of a cam gear and springs. The blow was estimated in this test at 150 lb.

Table 3—Granulometric Analysis of Crushed Granite Used in Test Pavement

Sieve	Per cent retained, by weight	Sieve	Per cent retained, by weight
1¾ in.	0.0	No. 2	29.5
1¼ in.	35.0	No. 4	7.2
¾ in.	28.3	Through No. 4	0.0

Per cent silt, 0; per cent voids, 45.4; specific gravity, 2.78

A revolution counter was used to record the number of complete revolutions of the shaft about the central pivot—both wheels passing completely around the path in one counted revolution. A picture of the machine will be found in the Engineering Record of Jan. 25, 1913, page 105.

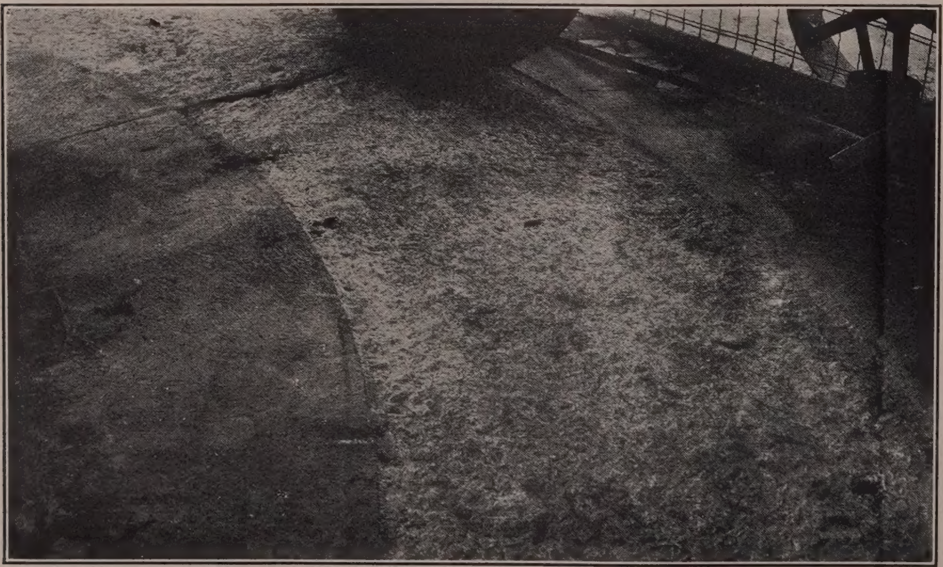
Table 4—Granulometric Analysis of Sand Used in Crushed-Granite Section of Test Pavement

Sieve No.	Per cent retained, by weight	Sieve No.	Per cent retained, by weight
4	0.0	50	5.7
10	18.3	80	8.9
20	30.7	100	1.9
30	18.5	200	2.0
40	12.8	Through 200	1.1

Per cent silt, 2.65; per cent voids, 30.0; specific gravity, 2.66; strength of 1:3 briquettes (average of 3), 3 days, 272 lb. per square inch, at 7 days, 350 lb.

Before the test was begun marks were placed on a circle inside the wheel track and levels taken at several points in the path to be traveled on radii through these marks. At the conclusion of the test readings were again taken on these points to determine the wear. These readings were made by members of the civil engineering faculty of the University of Michigan and will be used by them in a full report of the test to be rendered later.

After 19,315 revolutions the machine was stopped. The conditions of the various sections are shown in the photographs. The gravel concrete had worn badly and unevenly, as much as 1½ in. in places, while the granite concrete had shown only little wear, and that very even. The brick section laid on a mortar cushion was in excellent condition, while that on the sand cushion was beginning to go to pieces, as shown in the photograph. The darkening of the bricks in the photos is due to the fact that they had been wet before the pictures were taken.



Section (After Test) of Concrete Pavement Made with Crushed-Granite Aggregate

Extra Top Width for New Railroad Embankments

Proper allowances for shrinkage of new railroad embankments are difficult to estimate, on account of the unevenness of the shrinkage. The 1911 "Manual" of the American Railway Engineering Association states that "for green embankments shrinkage allowance should be made for both height and width," and gives a short table of percentages to be allowed for different materials and conditions. Nothing is said as to extra top width to allow for both the vertical settlement and the sliding out at the base, and in investigating the subject Prof. J. C. L. Fish, professor of railroad engineering at Stanford University, finds only two definite standards to cover this. One of these—the practice of the Michigan Central in 1902—was to add 1 ft. for each 5 ft. in height of embankment; the other, the Union Pacific reconstruction method, was to steepen the slopes until the top width at subgrade was the same as if the embankment had been built high enough to allow for full vertical shrinkage. In the August "Bulletin" of the American Railway Engineering Association Professor Fish proposes a method of determining the proper extra top width under any conditions, and the following notes are taken from his paper and the discussion on it by other engineers.

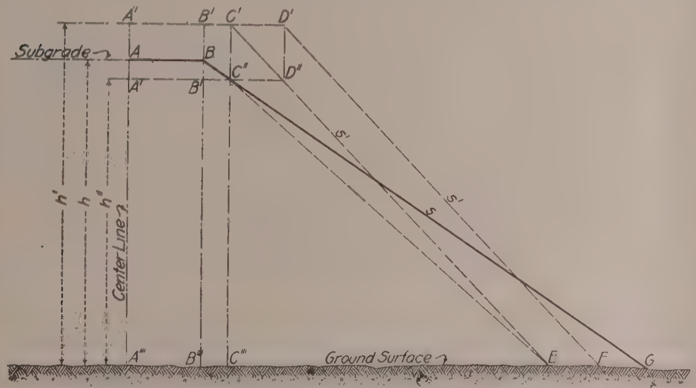


Fig. 1—For Fills on Level Ground

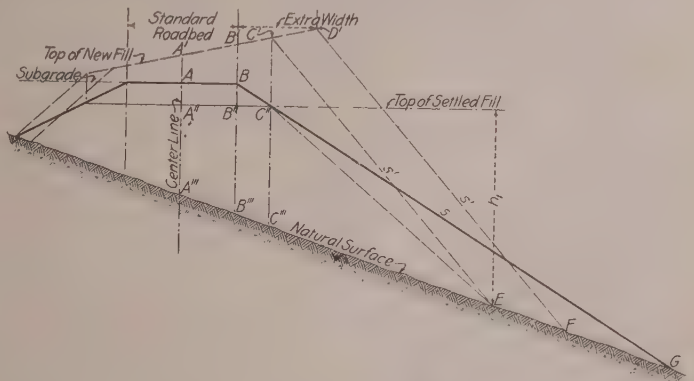


Fig. 2—For Sidehill Embankments

Fills may be built either on ground about level transversely or on a sidehill. They may be made with partial vertical allowance, with the expectation that they will settle below subgrade; with full vertical shrinkage allowance, so that they will settle just to subgrade, or with no vertical allowance, so that the top will drop below subgrade to the full extent of the vertical settlement.

Fills on Level Ground

For a fill on ground with little or no transverse slope Fig. 1 obtains. The subgrade height is h. The fill is built to the height h' with the expectation that it will settle to the height h''. The slope ratios are s for the settled fill and s' for the new fill, the ratio in each case being horizontal to vertical.

If the shoulder of the new fill is at B', the shoulder of the settled fill will be at B'', provided there is no sliding; but at the elevation of B'' the half width of the fill should be $A''C''$. Therefore the shoulder of the new fill should be carried out from B' to C', even ignoring sliding of the shoulder. With the shoulder of the new fill at C' the toe will be at E ($C'E$ being drawn with slope s'), and providing there is no sliding, the side line of the settled fill will be $C''E$. However, the shoulder will not be stable at C'' unless the triangle $C''EG$ is filled. To fill this triangle requires an additional slab of material, $C'D'FE$, on the side of the new fill. The area of the slab, settled, is $h'' \times C'D'$, and this must be equal to the area of the triangle $C''EG$. Therefore

$$h'' \times C'D' = \tfrac{1}{2} h'' \times EG,$$

or

$$C'D' = \tfrac{1}{2} EG = \tfrac{1}{2} (C'''G - C'''E) = \tfrac{1}{2} (h''s - h's') \quad (1)$$

The total extra top width to be added to the standard roadbed width on each side of the fill is therefore

$$B'D' = B'C' + C'D' = (h - h'')\, s + \tfrac{1}{2} (h''s - h's') \quad . \quad (2)$$

since $B'C' = (h - h'')s$.

For the fill which is built with full shrinkage allowance $h'' = h$, $h - h'' = 0$, and equation 2 reduces to

$$B'D' = \tfrac{1}{2} (hs - h's') \quad (3)$$

For the fill which is built new just to subgrade $h' = h$, and equation 2 becomes

$$B'D' = (h - h'')s + \tfrac{1}{2} (h''s - hs') \quad (4)$$

In practice h and h' are known and s' can be measured readily, but, owing to the fact that the vertical settlement and final slope of the fill must be estimated, the values given to h'' and s are subject to some uncertainty. Hence the engineer must use the foregoing formulæ with judgment, increasing the computed extra top width if the material is abundant or treacherous action in the fill is expected.

Fills on Sidehills

In order that the top of a fill that is built on a transverse slope may be level at the end of settlement the top of the new fill must have some slope in the direction contrary to the slope of the ground. Fig. 2 is drawn for a case in which the new fill is made with partial shrinkage allowance for vertical settlement. Since the fill is higher on one side than the other the extra top width required will not be the same for the two shoulders. To find the extra top width required for the shoulder on the downhill hide of the fill, proceed as follows: Plot A' on the cross-section to represent the top of the new fill on the center line; plot A'', $A'A''$ being the vertical settlement expected on the center line; draw a horizontal through A'' to cut BG at C''; plot C' vertically above C'', making $C''C' = C''C''' \times A''A'/A''A'''$; draw $C'E$ through C' with the slope s' of the new fill; find, by scaling, the approximate (settled) height h_1 of the slab $C'D'FE$ required to fill triangle $C''EG$, and find the area of the triangle $C''EG$.

The required width of the slab $C'D'FE$ is (approximately)

$$C'D' = (\text{area of triangle } C''EG)/h_1 \quad (5)$$

and

$$B'C' = AA''s \text{ (approximately)} \quad (6)$$

Therefore the extra top width to be given the downhill side of the fill is (approximately)

$$B'D' = AA'' \times s + (\text{area of } C''EG)/h_1 \quad (7)$$

The extra top width on the uphill side of the fill is found in the same way. When the new fill is to be built with full shrinkage allowance for vertical settlement, or to subgrade, the general method given above is used to determine the extra top width required at each shoulder.

The results obtained with the formulæ should not be used blindly, but with judgment, Professor Fish says in conclusion.

Discussion

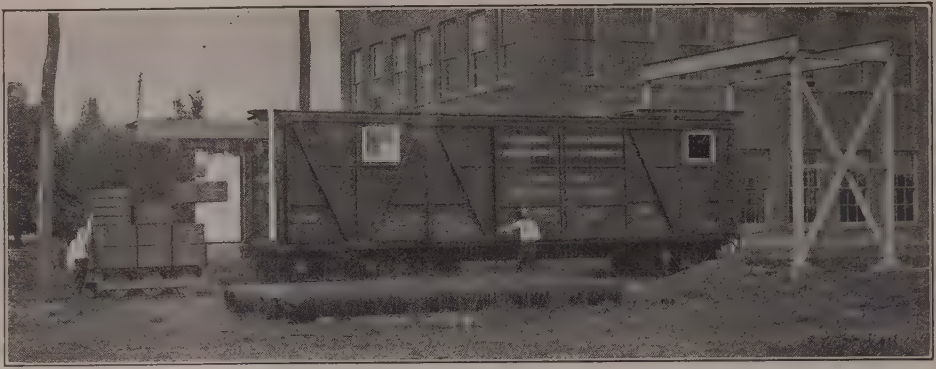
The paper was submitted for criticism to members of the committee on roadway of the American Railway Engineering Association, and the objection seemed to be general that the irregularity with which embankments shrink and the consequent uncertainty of some of the factors would render the formulæ of little practical value. Mr. F. L. Wheaton, of the Delaware, Lackawanna & Western, thinks that so much depends upon good judgment, even in the use of the formula, that the whole matter should be left to the engineers in charge. Mr. R. C. Falconer, of the Erie, expresses a similar opinion.

Mr. H. J. Slifer, consulting engineer, thinks that no engineer would deliberately make a new fill with the idea that it should settle below subgrade, and that, therefore, the formulæ assuming this condition should be eliminated. On the other hand, Mr. J. E. Willoughby, of the Atlantic Coast Line, has been obliged so many times to cut down fills that failed to shrink as expected that he has abandoned the practice of finishing high fills above subgrade except where the grades are less than ruling. On ruling grades he places enough material above subgrade to insure that when the time comes to finish the roadbed to receive the ballast the fill will be of sufficient height to finish the roadbed to exact subgrade elevation, all projection above this elevation being cut away. He finds that an extra width at subgrade of about 10 per cent of the depth of fill is usually satisfactory.

Answering these criticisms, Professor Fish points out that the diagrams are much exaggerated for clearness; that the method he proposes he used for four years as engineer on construction with the Lake Shore & Michigan Southern on fills as high as 120 ft.; and that while it is true that judgment must be used and certain of the factors must be estimated, it is easier to estimate these factors and work out the results than to estimate the results direct, just as it is easier to estimate heights and lengths of fills than to estimate their volumes. He contends that the slope s of the final settled fill can usually be closely predicted by comparison with other settled fills, while, on the contrary, comparatively few engineers know the original dimensions of settled embankments.

Scale-Testing Car of the Bureau of Standards

By A. H. Emery, Jr., Stamford, Conn.



Scale-Testing Car with Extended Crane Bridge, Weights and Truck

Ever since its inception the Bureau of Standards of the Department of Commerce has spent a great deal of time in studying the weights, measures and scales used in this country. The appropriations made last winter by Congress included an item for the study and testing of the railroad-track scales of the country, and the bureau has just put into service its first test-weight car. The equipment, including the large standard weights, was constructed by the writer, who also designed the Watertown testing machine.

The car was designed to meet the requirements given by Mr. L. A. Fischer, chief of the division of weights and measures of the Bureau of Standards, and Mr. C. A. Briggs, of the bureau, who has charge of the car.

The general idea of the apparatus is, briefly, a car carrying a large number of standard weights and apparatus for putting them out on the track-scale to be tested. A crane bridge which can be projected out of one end of the car is equipped with a trolley and hoist. The car carries a motor-driven truck, 90,000 lb. of large standard weights, and 10,000 lb. of weights in 50-lb. standards. The crane bridge, trolley and hoist, and the truck are all operated from a gasoline-motor and electric-generator set carried in the car.

In use the car is placed with the open end near the scale to be tested, the crane bridge is projected, and the truck is lifted out of the car, placed on the rails, run on the scale, scale readings are taken, and the truck is run back under the crane.

Most track scales have four pairs of first levers, and the scale is tested by observing the scale reading for a known amount of standard weights on the scale, testing it with the load applied over each set of levers. After taking the first set of readings any desired amount of weight is placed on the truck and the truck again run across the scale and returned and additional weights placed on it and more readings taken, thus checking the accuracy of the scale at a number of points within its capacity. This test is made in a short time and is not dependent on the accuracy of any apparatus except the standard weights.

Equipment of Car

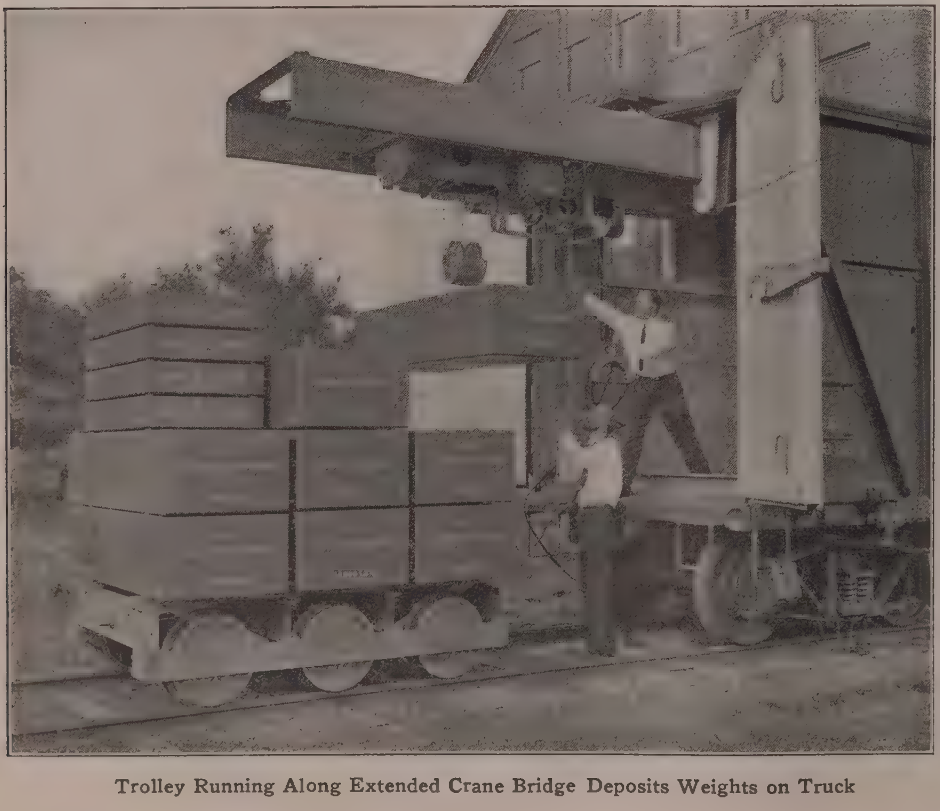
The car is of a standard heavy type with steel underframe, roof and doors, and steel-braced sides. One end is closed and heavily reinforced with steel and the other end has two large doors. A partition extends across the car about 10 ft. from its inner end, forming a room which contains a 10-kw Sturtevant generator set to provide power for operating the car and lighting it, a Davenport bed, water tank, wash bowl and toilet, stove, chair and folding table. The generator, radiator and fan are carried in the other part of the car, as is also the storage battery, which is charged from the generator set and is used for lighting the car when the generator is not running.

A frame for carrying the crane bridge is built into the car at the open end. Two H-beams and two channels form the bottom and side members respectively, and two other H-beams, bent to conform to the shape of the roof, form the top member, to which are fastened two castings which carry the wheels on which the crane bridge runs. The need of this heavy construction is apparent when it is considered that under certain conditions the load on these two wheels may be 20,000 lb.

At the partition there are two vertical 6-in. ship channels which carry castings and wheels for supporting the end of the crane bridge and for holding it down when the overhanging load is great enough to overbalance its weight. The bridge is projected by means of a motor-driven screw fixed to the rear end of the car and a nut fastened to the side of the crane.

The uprights at the partition are stayed to the rear end of the car by long rods, and the crane bridge can be bolted to these uprights for shipment. The trolley also can be bolted to the bridge. The truck is provided with a 5-hp motor with worm gearing and has a capacity of 50 tons. It is connected to the generator by a cable. This truck weighs slightly more than 5000 lb.

The weights, of which there are eight of 10,000 lb. each and four of 2500 lb. each, are of cast iron, machined all over, painted, and adjusted to exact weight. The probable error of the whole 90,000-lb. set is ± 0.24 lb. and the average probable error of any one 10,000-lb. weight is ± 0.08 lb. The 10,000 lb. of 50-lb. weights are cased in three boxes equipped with special handling apparatus so they can be easily removed from the car to a wagon and run about any town where the car may be located.



Trolley Running Along Extended Crane Bridge Deposits Weights on Truck

Lighting Principles in Mills

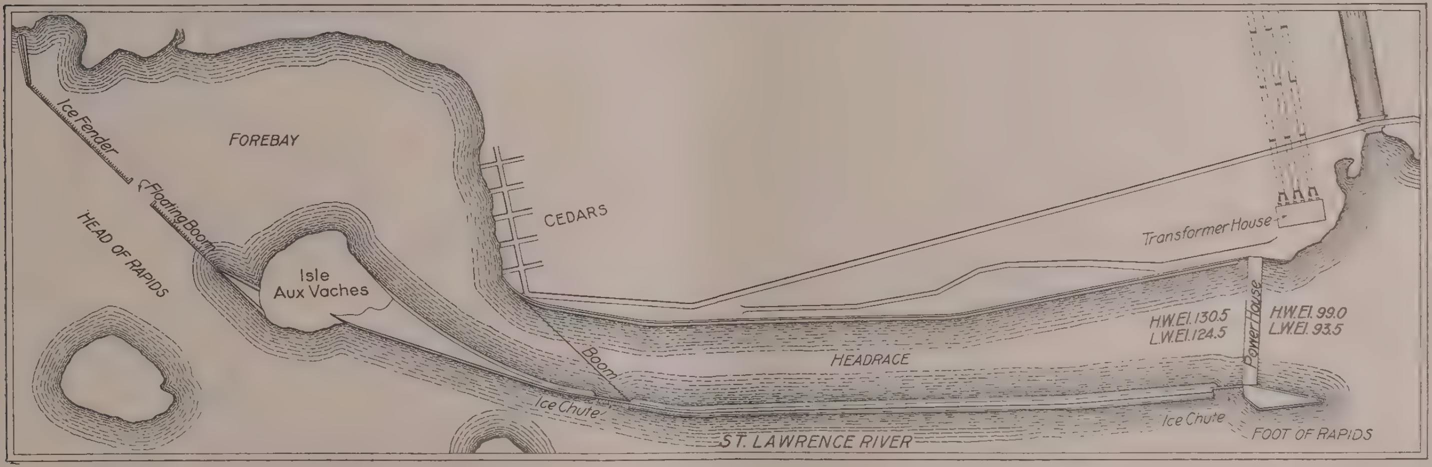
The following fundamental principles of cotton-mill lighting have been outlined by Mr. Albert L. Pearson in a paper read before the American Society of Mechanical Engineers: As the operative is the one for whom the light is provided, he should be given every consideration. Adjustable lamps, hung low down, tempt operatives to meddle with them. Lamps should be arranged to give uniform illumination at the working plane, avoiding shadows, as far as possible, and particular attention should be given to the requirements of each machine. White walls and ceilings add to the effectiveness of the lighting system. With individual driving of machines it is possible, owing to omission of overhead belts and shafting, to keep the rooms cleaner than with mechanical or group driving, thus benefiting the lighting system. On account of glare, low exposed units should be avoided. In places where it is necessary to have the lamps low down, reflectors which will entirely conceal the filament of the lamp should be used. The position of lamps should be carefully determined, both as to spacing and mounting height. In general, the height of the lamp above the floor should be such that with the spacing available the lines representing the angles of maximum illumination with a given type of reflector will cross at the working place.

Cedars Rapids Hydroelectric Development

Eighteen 10,800 Horsepower Turbines, Receiving St. Lawrence River Water under 30-Foot Head, Will Generate Current for Electrochemical Purposes

At Cedars Rapids, about 8 miles above Vaudreuil, Quebec, in the St. Lawrence River, construction has begun on the canal and power house for the Cedars Rapids Manufacturing & Power Company, of Montreal. This plant when completed will be one of the largest power developments on the St. Lawrence River and will be by far the largest development in Canada. One of the conditions under which the plant is being constructed is that it must not in any way interfere with the navigation of the St. Lawrence River, a condition which, of course, prevented the building of a dam across the stream.

In the first installation not all of the power will be developed. Ten 10,800-hp main units and three 1500-hp exciters are being installed at the present time. Ultimately the plant will have eighteen main units and six exciters, giving a total electrical output of 160,000 hp. It is expected that most of the power will be used for electrochemical purposes. The Montreal Light, Heat & Power Company and the Aluminum Company of America have already contracted for large amounts of this power.



Plan Showing Relation of Power House and Transformer House to River, Headrace, Forebay and Ice Fender

General Conditions

At Cedars the St. Lawrence River is about 3000 ft. wide and has a fall of about 32 ft. in 2 miles. Careful hydrographic investigations have been made on the St. Lawrence watershed, and very extensive work has been done by both the Canadian and United States Governments to determine the total flow of the river, the drainage basin of which is not less than about 350,000 sq. miles. Of the flow in the river the Canadian Government has granted to the power company the right to use a maximum of 56,000 sec.-ft.

There are three rapids in the St. Lawrence River within a distance of about 15 miles—the Coteau Rapids, the Cedars Rapids and the Cascades Rapids. These rapids are navigable by the Soulanges Canal, which starts at Lake St. Francis above the Coteau Rapids and ends at the Cascades at the junction of the Ottawa and St. Lawrence Rivers, just below the Cascades Rapids. Under an old agreement with the Government, water is now taken from the Soulanges Canal for the development of about 10,000 hp by the Provincial Light & Power Company, a subsidiary of the Montreal Light, Heat & Power Company, and power is taken from that station, which is located only about ¼ mile from the Cedars development, for use as motive power for construction purposes.

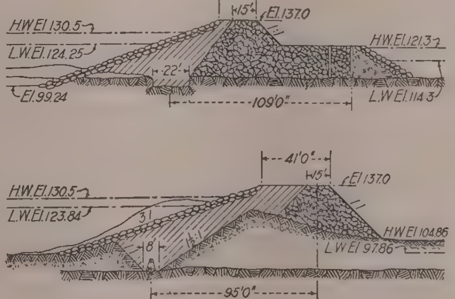
In front of the town of Cedars, at the upper end of the rapids, the river forms a bay, at the upper edge of which is located the Isle aux Vaches. This point has been chosen as the intake for the power canal, 10,000 ft. long. Above this island will be located ice fenders to prevent floating ice from Lake St. Francis entering the canal.

From the Isle aux Vaches an earth dike will be built extending approximately 10,000 ft. to the power house and running parallel to the north shore of the river. In this dike are located two spillways, each 300 ft. long. The first of these spillways is situated about 3000 ft. below the Isle aux Vaches and the second immediately above the power station.

By means of booms it is expected to divert any floating material, ice, etc., over these spillways. The canal proper varies in width from 700 ft. at the intake to about 1200 ft. at the power house, and the depth of water will vary with the stage of the river from 28 to 34 ft. The velocity of the water at the lowest stage of the river will be about 4 ft. per second. The canal is designed for 56,000 sec.-ft.

From the first or upper spillway a concrete core wall, keyed in rock, is constructed in the earth bank, forming a cutoff to prevent seepage. The north side of the canal will be the natural shore of the river, which will be straightened to form one bank of the canal.

For the full development there are about 3,500,000 cu. yd. of earth and rock to be excavated. The main work on the canal consists of removing two peninsulas, one located near the intake and another just above the power house. Nearly all excavation is taking place in these two peninsulas.



Typical Sections of Canal Wall

The power house will have ultimately a length of 1184 ft.; of this about 700 ft. are being constructed at the present time. The main units are grouped in three groups of six. Between these groups are located the exciters, three in each bay. In the exciter bays are also located ice chutes, so that ice can be passed without conveying it to the spillway. The total width of the power house is 132 ft. The main units are spaced 50 ft. center to center.

Main Units

The turbines are guaranteed to generate 10,800 hp under a head of 30 ft. They are to be of vertical-shaft, single-runner Francis type. The turbines built by the I. P. Morris Company are designed for a speed of 55.6 r.p.m. and those built by the Wellman-Seaver-Morgan Company are designed for 54.3 r.p.m. The experimental model of the I. P. Morris runner, tested at Holyoke, gave a maximum efficiency of 90.05 per cent at eight-tenths gate opening.

The sixteen-bucket cast-iron runner has a rated diameter of 11 ft. 10 in. and a discharge diameter of 17 ft. 7 in. It will be of the built-up type, cast in four sections, each having four buckets. These sections will be bolted together and connected to a cast-steel crown plate, which is made in one piece. This crown plate is in turn bolted to the flange of the 24-in. shaft. At discharge diameter the runner sections are bolted to a cast-steel retaining ring, thus making an exceedingly rigid construction.

The shaft is held in place by two guide bearings, the thrust bearing being located above the generator. The lower guide bearing, located in the head cover of the turbine immediately above the runner, is about 8 ft. long, and is lined with lignum vitæ and provided with water lubrication.

The upper guide bearing, situated immediately below the thrust bearing, will be babbitted and lubricated by oil under pressure.

Thrust Bearing

The thrust bearing carries the combined thrust of the revolving parts of the unit, amounting to approximately 550,000 lb. This bearing, 5 ft. in diameter, will be of the Kingsbury type (see Engineering Record, Jan. 11, 1913, page 44, and Jan. 18, 1913, page 75), with a roller auxiliary. The shoes of the bearing are babbitted and supported on the head of 5-in. jackscrews, thus facilitating easy adjustment, and replacing the wedges and shims previously used in this type of bearing. The roller auxiliary is so arranged that should the babbitt facing on the shoes burn out and allow the revolving parts to drop, the thrust block on the shaft will bear on the upper washer of the roller bearing and so carry the revolving element.

Above this bearing is a cast-steel thrust block, 3 ft. 4 in. high, in which the shaft is held by means of one circular and one longitudinal key. The upper parts of these keys and the shaft are kept in place by a circular notched ring capscrewed to the top of the thrust block.

As it is absolutely necessary for the Kingsbury bearing to be submerged in oil, the housing is provided with a 2½-in. overflow pipe, maintaining the oil level 3 in. above the bearing surface. The housing incloses the upper parts of the shaft, and the bearing has windows above the highest oil surface and is illuminated with electric lamps.

The thrust bearing support, consisting of a central twelve-faced cast-iron plug, containing the upper guide bearing, is above the generator. To the faces of the plug are bolted twelve brackets, extending over and supported on the generator frame. This spider-shaped support has a span approximately 37 ft. long.

Speed Ring

The speed ring is of cast iron and is made in five sections, each of which contains two heavy crescent-shaped vanes for the distribution of water from the scroll case to the twenty cast-steel wicket gates. This speed ring is so designed that it will transmit the weights of the generator and of the concrete substructure above the scroll case, together with the thrust and weight of the revolving parts of the unit, to the concrete foundations below the turbine. In erecting the unit this speed ring will first be bolted together, leveled and adjusted by means of ten jackscrews bearing on steel supports, and then incased in concrete.

The cast-steel operating ring manipulating the wicket gates will be directly connected to two 22-in. regulating cylinders having a stroke of 20 in. These cylinders are equipped with trunk pistons, no crossheads being required. The ring itself is of a hollow triangular cross-section and turns on bronze guides attached to the head cover. These guides are lubricated by grease from compression cups. The wicket-gate levers are connected with this ring by cast-iron links, while pockets in the cast-iron pit liner house the wicket-gate regulating cylinders. The parts between the head cover and the generator are accessible by a stairway from the main generator floor of the power station.

The total height of the unit from the bottom flange of the speed ring to the top of the shaft above the generator is approximately 42 ft. 3 in. The depth of the substructure from the generator floor to the same flange is 25 ft. 6 in.

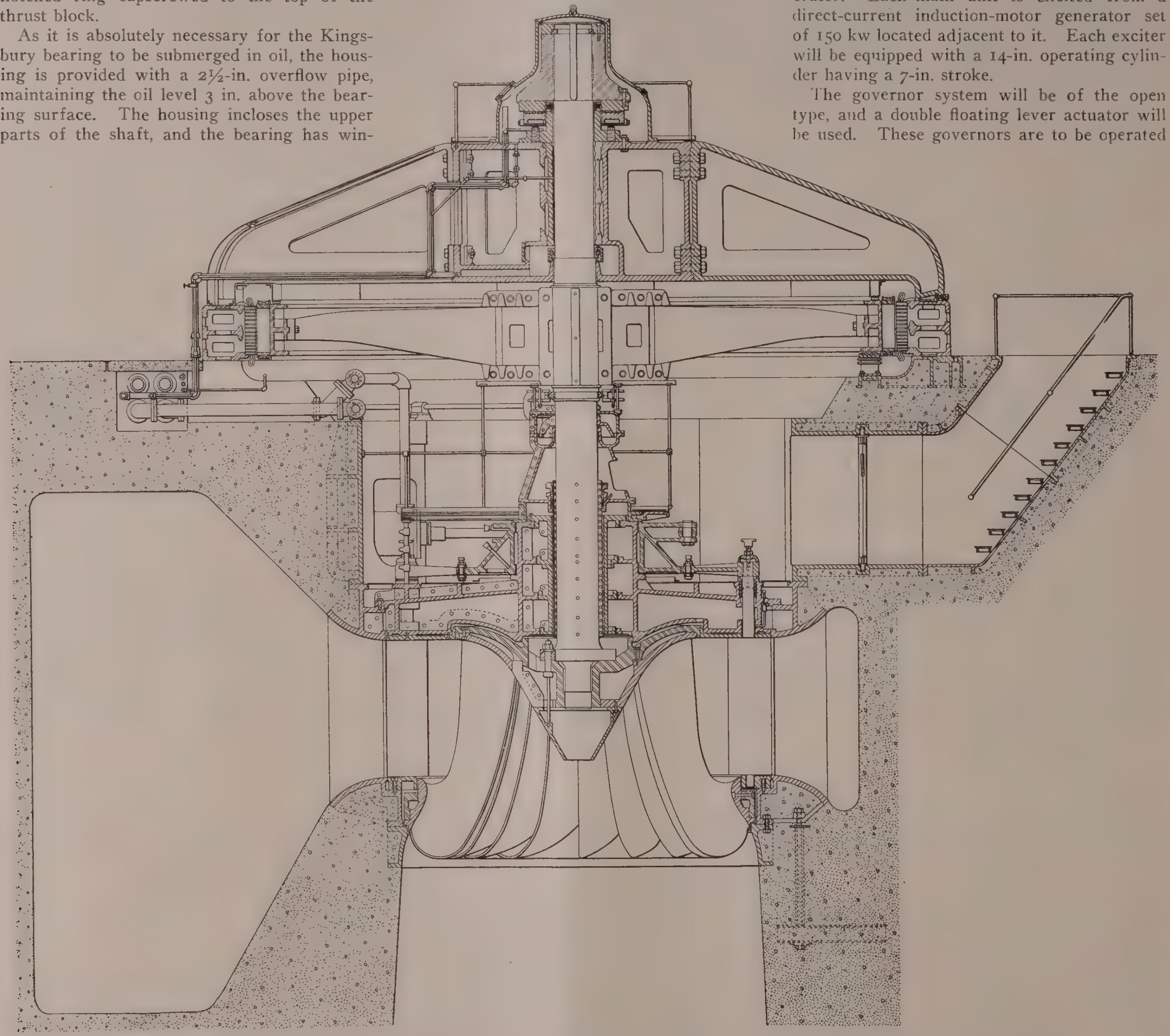
Exciters

The design of the exciter units will follow the same general lines as the main wheels. Each of these machines will have a capacity of 1500 hp at 150 r. p. m. and be connected to a 60-cycle, 2200-volt alternating-current generator. Each main unit is excited from a direct-current induction-motor generator set of 150 kw located adjacent to it. Each exciter will be equipped with a 14-in. operating cylinder having a 7-in. stroke.

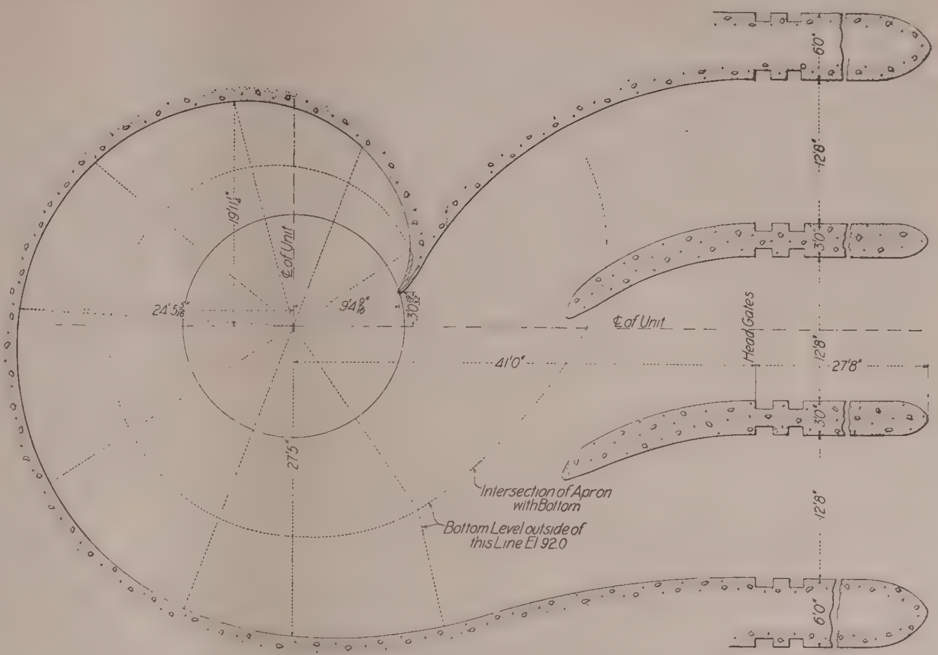
The governor system will be of the open type, and a double floating lever actuator will be used. These governors are to be operated with water at 200 lb. pressure, received from the central pumping station located in one of the exciter bays.

In this pumping plant there will be installed four motor-driven centrifugal pumps, each having a capacity of 1100 gal. per minute. Two of these pumps are able to supply the first installation of ten units and three exciters. The water will have a pressure of 200 lb. and be delivered through mains to accumulator tanks, one for each main unit. Air for these tanks will be supplied by motor-driven compressors in the pump chamber.

The return water from the governors is discharged into concrete trenches running the entire length of the power house and placed in a tunnel on the upstream side of the units. These trenches, which are of liberal dimen-



Cross-Section of 10,800-Horsepower Vertical-Shaft, Single-Runner Turbine for the Cedars Rapids Hydroelectric Plant



Plan Indicating Shape and General Arrangement of Scroll Case Wheel Chamber

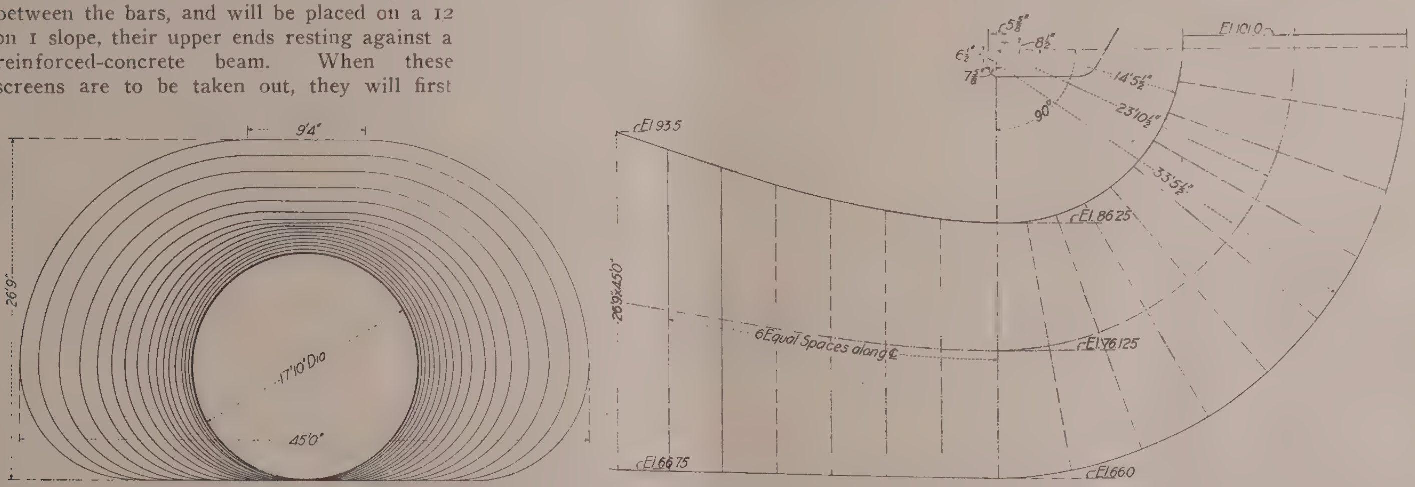
sions, will also act as sump tanks from which the pumps will draw their water. As these return conduits are open to the atmosphere, there will be no back pressure on the governors on returning the water to the pumps.

The central pumping plant will also include motor-driven oil pumps for raising the oil in the tanks located in the upper part of the building. These tanks will supply oil by gravity to the thrust and guide bearings.

Wheel Settings

Scroll cases molded in concrete will convey and evenly distribute the water to the speed rings of the units. As the roof of this intake is located only 4 ft. below the low-water surface in the canal and air must be prevented from being drawn to the wheel chamber, the entrance velocity of the water has been chosen at 3.5 ft. per second. The accompanying plan shows the shape of the wheel chamber. It will be closed by means of three steel gates of 12 ft. 8 in. free span, divided into an upper and a lower section. At the junction of these sections there will be a reinforced-concrete beam to serve as a support and as a seal. The gate sections will be provided with separate slots so that they can be manipulated independently.

The screens will have a 4-in. clear opening between the bars, and will be placed on a 12 on 1 slope, their upper ends resting against a reinforced-concrete beam. When these screens are to be taken out, they will first have to be tilted to a vertical position and then hoisted by the overhead crane.



Section and Longitudinal Elevation Showing the Shape of the Draft Tube for the Cedars Rapids Plant

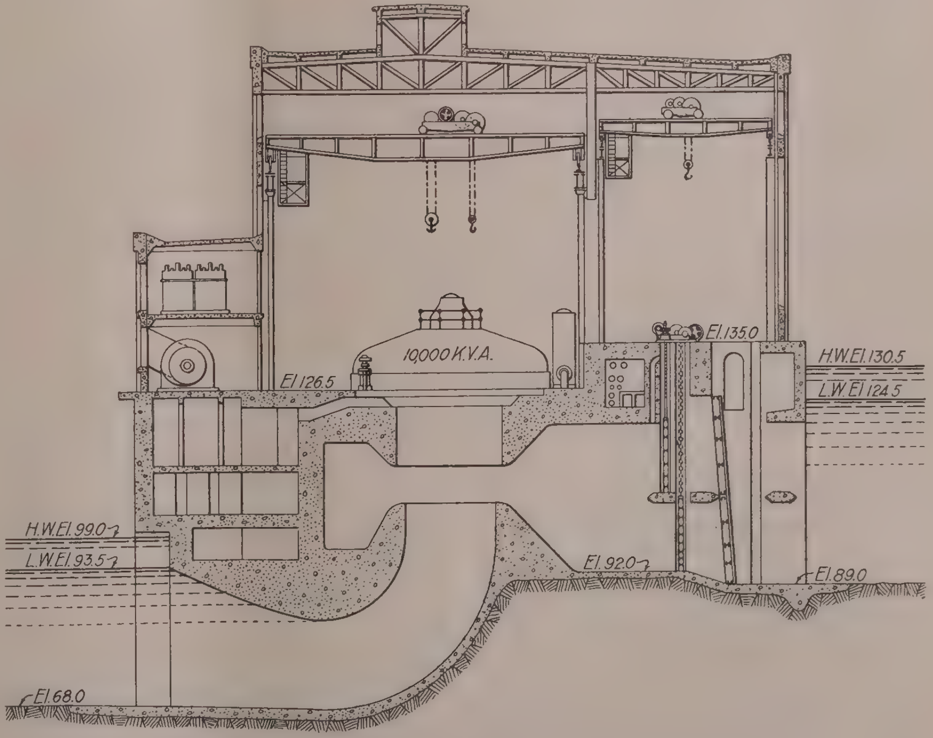
The concrete partitions between the units are 6 ft. thick and the piers between the gates are 3 ft. All of these partitions and piers are reinforced and braced together by means of reinforced-concrete beams.

The upper beam serves as an ice fender, and at the same time distributing to the substructure the loads from the upstream wall of the power house.

In the accompanying drawings are shown the general arrangement and cross-sections of the draft tube. Abrupt velocity changes have been avoided in the retardation of the water from 15 ft. per second to 3.5 ft. As shown, the cross-sections are symmetrical around the vertical but not around the horizontal center lines.

Superstructure

The superstructure will be of reinforced-concrete built by the unit method, the contract having been let to the Unit Construction Company, of St. Louis. It will be provided with steel window sashes and metal doors. There will be, longitudinally, three divisions of the building—gate room, generator room, and auxiliary apparatus room. The last room has two floors—the lower for governor pumps, oil pumps, blowers, motor-generator exciter sets, etc.; the upper for low-tension buses, switch-



Sectional Elevation of Power House on Center Line of Main Unit boards and low-tension oil switches. The three bays opposite the exciters will have a third floor above the bus room for oil tanks.

The transformer house will be of a separate structure, since this will expedite construction as against a design placing the transformer equipment in the main building. Under the adopted plan two separate organizations will be engaged at the same time.

The generators will be 10,000-kva, 6600-volt, 60-cycle, 3-phase machines, and the outgoing current will be stepped up to 110,000 volts. The outside diameter of the generator stator is 37 ft. 4 in.

Personnel

The property was taken over by the financial interests with which the Shawinigan Water & Power Company and the Montreal Light, Heat & Power Company were allied in January, 1912. Up to that time the property was controlled by Mr. D. Lorne McGibbon, who remains as one of the directors of the company, and the engineering work had been in charge of Mr. Henry Holgate, consulting engineer, of Montreal.

After the transfer of the property the following officers were elected: J. E. Aldred, president; Howard Murray, vice-president, and J. S. Norris, secretary-treasurer.

The engineering work has been divided between Mr. R. M. Wilson, chief engineer of the Montreal Light, Heat & Power Company, who has charge of the electrical engineering, and Mr. Julian C. Smith, chief engineer of the Shawinigan Water & Power Company, who has charge of the hydraulic engineering and construction work. Mr. W. Dan Bergman is Mr. Smith's first assistant in charge of design, and Mr. G. P. Hawley is resident engineer.

The problems of design and construction were then actively taken up, and an effort was made to contract the work. At that time comparatively little was known regarding the ground conditions, particularly as to what would be encountered after the site was unwatered. On account of these uncertainties it was decided that the company would undertake the preliminary work, and accordingly a construction force was organized and the cofferdams were built. There are about 15,000 lin. ft. of cofferdams, which were practically completed in June of this year.

As soon as the unwatering was accomplished another effort was made to contract the work, this time with success, and Fraser, Brace & Company, of New York City, were given the contract for all excavation and for the concrete work in the power house substructure.

A contract was awarded to the I. P. Morris Company for the construction of nine wheels, of which three will not be installed until a later date, and for the governing apparatus and exciter wheels. The Wellman-Seaver-Morgan Company was awarded a contract for three units of practically the same design, although the speed is different, as previously stated. The General Electric Company has the contract for twelve main generators. The contract for the generators does not include the thrust-bearing supports. These supports will be built by the turbine manufacturers.

It is expected that energy can be delivered to consumers at the end of 1914.

Concrete Resistors for lightning-arrestor service, as used by a Georgia power concern, are made of concrete blocks 4 ft. long and 1 sq. ft. in cross-section, set upright at the point of installation. Two squares of bronze or copper mesh cast into the blocks near either end connect with the source of energy and with a ground wire.

Fires in the Rocky Mountains Forest Reserve

The Rocky Mountains Forest Reserve lies just north of the international line in the Province of Alberta and embraces an area of about 18,750 sq. miles of timber land, in which lie the sources of most of the streams flowing through Alberta and Saskatchewan. Since the reserve was established in 1910 there has been an organized effort to check the forest waste by fire, which is said to have been out of all proportion to that suffered by other forest areas in the West. Early explorers found heavy timber over hundreds of miles of country on the east slope of the mountains, which is now practically denuded, and it has been found, states Mr. W. N. Miller, district inspector of forest reserves, in a paper delivered before the Western Canada Irrigation Association, that at least 90 per cent of the forests in this section are not one hundred years old, while probably 75 per cent are not over fifty years old. In other words, evidence points to a surprisingly heavy and widespread destruction of the forests within the last fifty years, during which time travel in these mountains has been a factor of importance.

In providing for fire protection the Rocky Mountains Forest Reserve has been divided into five sections, each of which is under the control of a forest supervisor. These divisions run from 1,000,000 to 3,000,000 acres in area and are laid out on topographical lines so that they can be administered practically independent of each other. Each supervisor further subdivides the district under his charge into ranger districts, which average roughly about 200,000 acres each. The ranger districts are also bounded by mountain ranges or other topographical lines, and the ranger in charge of each is made responsible for all work within the district. Besides fire protection duties he is charged with the administration of timber, the construction of trails and bridges, and other supervisory work. It is thus possible to provide year-round employment, which is said to secure a better type of ranger and keep men in the service who are familiar with the districts and the work involved.

The success of the fire-prevention work, Mr. Miller points out, has been largely due to developing the lookout system and perfecting prearrangements for despatching help in emergencies. The patrol system would be wholly inadequate under the conditions in the reserve, but the plan of locating fires by triangulation from lookout peaks and by communicating by wireless or telephone has made it possible to secure adequate protection, Mr. Miller states, at a cost of from 1 to 2 cents per acre.

Railroad Bills to the Number of 1395 were introduced in forty-two State legislatures in session in 1913, according to a report of a special committee of the American Railway Association. Of these, 230 became laws. They covered such subjects as better protection for employees, arbitration, train rules, equipment, terms of employment for those in railway service, block and other signals, clearances, crossings, maintenance of way, equipment of stations, hospitals and relief departments, claims, trespassers and accident reports. Fourteen out of 107 "full-crew" bills were passed, and nine out of 100 regulating terms of employment became law. Of sixty-five bills to prohibit trespassing on railroad property, none was passed.

Rebuilding Humboldt Bay Jetties

Entrance to Humboldt Bay, in northern California, is between two low sand spits, and in order to maintain a channel jetties were built on each side many years ago. For some time these jetties had been settling and disintegrating in part, and thereby failing to keep the channel open. In 1911 the reconstruction of one of the jetties was begun. The work is described by Mr. Morton L. Tower, resident engineer on the work, in the "Professional Memoirs," of the Corps of Engineers, U. S. A., and the Engineer Department at Large. These notes are taken from the description as printed.

In planning the work it was desired to use large-sized stones, lowering them to place with a crane to avoid breaking them. The rock had to be brought 7 miles, and it was decided to use standard equipment salable after use. For these reasons a low-level track on the jetty was determined upon, with a concrete cap to hold the track when the jetty was swept by waves and to furnish a thoroughly stable foundation for the crane.

The concrete cap and tracks cost as follows, per linear foot, including 1 per cent. of track lost by wave wash before the concrete had set:

Class 3 stone in voids of jetty enrockment, 4½ tons	$6.75
Class 4 stone, 2 tons	3.00
Cement, 0.95 bbl	2.00
Total concrete material	$11.75
Labor	2.86
Lumber, 50 ft. b.m.	.70
Fuel, oil, repairs, etc	1.36
Total (not recoverable)	$15.97
Rails and joints (recoverable)	2.13
Total	$18.10

The cost of plant was as follows:

One concrete mixer, complete with boiler and engine	$1,500
One flat car	600
Tanks and small tools	250
Labor assembling plant	200
Total	$2,550

Cost per linear foot of jetty, 53 cents.

In the method of construction used enrockment is first brought to an elevation averaging 2 ft. below the finished grade with Class 2 stone, pieces weighing from 1000 lb. to 10 tons. None of the pieces of Class 2 stone is allowed to project above 6 in. below grade—the bottom of the ties. Voids in the mass are then filled with Class 3 stone, pieces weighing from 3 to 500 lb., and the top is leveled off at from 8 to 10 inches below grade. Holes are choked by hand-placed stone. A rough form is made by tying wale pieces, 6 x 8-in. x 20-ft. ties, together with wires 5 ft. apart and nailing to them short vertical boards with the bottoms in contact with the rock, a rock dam being built at the front end of the form. The concrete is mixed rather dry and placed with a 1-yd. self-dumping and self-righting bucket, handled by the stone-unloading crane.

Concrete Plant

The concrete mixer is mounted on the end of a standard flat car, the discharge chute delivering over the end. Mixing water is supplied by gravity from a tank on the opposite end of the car. Oil fuel is supplied by gravity from a tank near the water tank, the oil and water tank supplying the stone-unloading crane. The oil is pumped and the water flows by gravity to the crane supply tanks. The cement for a day's operation is carried on the car. A Foote batch mixer of 21 cu. ft.

capacity of the end-discharge, steam-driven type, is used.

At the outer end of the jetty there are two working tracks. All material and equipment are of regulation master car-builders' pattern and dimensions. The tracks are 14 ft. 7¾ in. center to center. When in operation the concrete mixer car occupies the left-hand track and the car containing the aggregate the right-hand track. A working movable platform fills the space between the cars, leaving about 1 in. clearance over the stake pockets. The elevation of the platform is the same as that of the car decks. The aggregate is shoveled directly from the cars into the charging skip and a large barrow. The charging skip is marked by a row of rivets at the height containing the charge required. In order to work a sufficient number of shovelers to keep the depositing bucket in motion it was found necessary to provide a greater length than was possible by shoveling into the skip alone. A two-wheel barrow, running on rails and holding about 15 cu. ft. is mounted on the mixer car. This allows the charging shovelers to be distributed over the whole length of a standard flat car.

Concrete Crew

For depositing the concrete the most economical crew is eight laborers charging the aggregate, one laborer charging cement and one engineman operating the mixer. The concrete is placed, spread and tamped by the regular stone-unloading crew, consisting of engineman, four laborers and the foreman. This crew will generally build a section of track 18 or 20 ft. in length in 2¼ hours.

Under the specifications for the material the contractors are allowed to supply either broken stone, crusher run or unscreened river gravel for aggregate. The material furnished is tested for proportioning in the following manner: The aggregate as delivered is screened and the portion passing a plate containing ¼-in. diameter holes is considered sand and the remainder stone. If necessary, sand is added to form a mixture corresponding to a 1:2½:5½. The gravel supplied contains rather a larger proportion of sand than is required. When broken stone is delivered it is necessary to add about 16 lb. of sand per 100 lb. of aggregate as received.

Tests of the material delivered are made from time to time to form a reasonably close record of the proportions of the aggregate. This very rough system of measuring the material has resulted in a very fair concrete, the requirements of the work being considered. A more refined method of proportioning the materials, while perhaps resulting in a denser concrete, would render necessary screening and measuring plants and labor for handling, and it is doubtful if any real economy would be effected. In order to accelerate setting during severe weather when the fresh concrete is exposed to wave wash within a few hours after depositing, the cement portion is sometimes increased.

Concrete displacers—stones of from 3 to 200 lb.—are added to the mass below and between the ties. The larger portion of the stone supplied is a close-grained igneous rock, weighing about 198 lb. per cubic foot. It is very difficult to quarry, breaking into very uneven fragments. However, by proper manipulation, a minimum of waste is secured, and, as there is no covering soil to be contended with, the quarry is very satisfactory.

The work has been under the direction of Col. John Biddle, Lieut.-Col. Thos. H. Rees and First Lieut. Thos. H. Emerson, Corps of Engineers, U. S. A.

Garbage-Plant Appraisal in Chicago

Abstract of Opinions of the Appraisers, Principally on the Going Value of the Plant, about Which There Was Disagreement

Failure of three appraisers to agree on the fair value of the garbage disposal plant of the Chicago Reduction Company was due primarily to their different conceptions of the basis of valuation. The agreed facts and opinions of the appraisers, Col. Henry A. Allen for the city, Mr. Harold Almert for the company, and Mr. Leonard Metcalf, their joint choice, are given herewith, inasmuch as this is an unusual appraisal carried out under unusual restrictions. Comment on the result will be found in the editorial pages.

Agreement was reached on original cost, $386,035; and on gross reproduction cost, $296,370, including 7 per cent for engineering, 5 per cent for contingencies and omissions and 4 per cent for interest during construction.

The accrued depreciation, based on the gross reproduction, was estimated at 41 per cent by Colonel Allen, 24 per cent by Mr. Metcalf and 14 per cent by Mr. Almert. Relative to real-estate values, the company contended that substantially higher values than for ordinary manufacturing should be placed upon this property for the purposes of garbage reduction, by reason of its peculiar adaptability and location both as a center for collection and disposition and on account of its proximity to the stockyards district. Depositions were taken from parties interested in garbage-disposal ventures in other cities to indicate that the procuring and maintaining of a suitable site was a tangible asset. For this reason Mr. Almert raised the conceded value of 80 cents per square foot for factory purposes to $1.25 per square foot on the 150,000-sq. ft. area. Colonel Allen contended that no additional allowance should be made. Mr. Metcalf suggested $15,000 would be reasonable.

The development expense could not be ascertained from first-hand sources, a fire in 1912 having destroyed the company's books; but Mr. Almert stated that the company estimated it at $307,311. Colonel Allen was of the opinion that no additional recognition should be given to this element of value.

Brief statements of views of the different appraisers follow:

Colonel Allen took the position that "the city of Chicago is not compelled to purchase or to make use of the plant of the Chicago Reduction Company for taking care of its garbage in a sanitary manner. The city officials are in possession of data, based upon properly conducted experiments, conclusively proving it practicable to handle immediately the garbage of the city in a sanitary and inoffensive manner should the present plant be destroyed or otherwise become unavailable for use by the city. Further consideration should be given to the fact that there is a strong public sentiment favoring the abandonment of the reduction process and caring for and disposing of the city's wastes, including garbage, by means of incineration. The city council has already authorized the purchase of several tracts of land for the installation of incinerators," and that the property should be valued upon the basis of the fair value of the land for manufacturing purposes, and the value of the plant from the point of view of its temporary utility to the city. Upon this basis he valued the property at $208,582.

Mr. Almert valued the property on the basis of its income in the light of its past history at $750,000.

Mr. Metcalf based his finding on the preamble of the contract, in which is stated:

"The appraisal is to be made upon the plant as a going concern, taking into consideration the reasonable development expenses incurred by said company in bringing said garbage reduction plant to its present condition, with all of the natural accretions thereto, and is to include the lands upon which said plant is now located, taking into consideration the peculiar adaptability for the purposes for which it is now used, with all of the appurtenances thereunto both above and below ground, buildings, machinery and all tools, stores, supplies and merchandise on hand, and all assignable outstanding contracts."

In Mr. Metcalf's opinion, therefore, the property should be valued as one having a developed business, or one which was producing from garbage by-products which found ready market at prices showing profit to the company, independent of any payment to the company by the city for the services rendered; and that under the terms of this agreement it was unfair to assume that the property was to be valued upon the assumption that it was to be abandoned by the city, or to be utilized merely for its temporary convenience.

Mr. Metcalf called attention to the fact that in his basis of valuation Colonel Allen had not, in his judgment, met the conditions of the contract, and that, even from the city's point of view, he had overlooked the important element of value that the present plant could be operated by the city temporarily at a profit, until its own plant could be built, and still sell the existing plant at a price, in Mr. Metcalf's judgment, in advance of the sum of $208,582, the approximate value suggested for this plant by Colonel Allen.

Mr. Metcalf pointed out, on the other hand, that, in his judgment, Mr. Almert had not recognized that the company was not entitled to consider in its earnings the amounts heretofore paid to it by the city, as there was every indication, not alone in the local situation, but in similar situations elsewhere in this country, that the city would not pay so substantial a sum hereafter for the reduction of its garbage.

Mr. Metcalf called attention also to some of the differences in conditions under which such a company as this operated, as compared with the ordinary public utility company, to wit: that the market for its products already existed and was in large measure independent of the operation of garbage reduction works; that the company did business indirectly with the individual instead of directly; that the company offered substantial inducement to the householder for his garbage by relieving him of any expense incident to its collection or transportation, whereas the ordinary public service corporation furnished its service at actual and substantial annual cost to the consumer; that the development or going value period was likely to be brief, and its amount therefore relatively small as compared with those of the ordinary public utilities; that the hazards of such a business were unusually large in fact and in the minds of those entering into these projects; and finally that, while the property of the ordinary public utility is likely to be used only for the service for which it was acquired by the utility, in the case of such a plant as this the property might well be converted to and have substantial value for other uses than those for which it was originally designed, such, for instance, as the rendering of certain portions only of the garbage, the rendering of other materials than garbage, the making of fertilizers, or the development of other chemical or manufacturing processes for which substantial parts of the existing plant might be reasonably well adapted; and that, being located upon private property, the plant could be put to such uses, provided the work done by it complied with the regulations of the city and the legal requirements which might be applicable to it. Therefore, to fail to recognize this fact, as would be done in considering solely the temporary utility and scrap value to the city of parts of the plant, would be to deprive the company of a portion of its property without just compensation.

Going Value

From Colonel Allen's arguments on "going value" the following notes are abstracted:

Even in appraisals of public service corporations or public utilities there has not been a well-defined and generally accepted definition of the term "going concern," nor a consensus of opinion as to the methods to be used in establishing its value.

In appraising going value reference must be had to the stability of the enterprise, because without stability or surety the going concern value will be speculative, and hence of uncertain or no value. A business may be established, but owing to failure of outside contract fulfilment, uncertainty of supplies, or other conditions, it may be deprived of important and necessary trade resources. Such a case would be a sawmill which, due to forest fires, has been deprived of its source of timber supply.

In the case of the Chicago Reduction Company, on Aug. 31, 1913, the contract upon which the "going concern" value as such existed, expired. Hence, that which brought the company into existence, permitted it to exist, enabled profits to be made, and gave it a positive going concern value ceased. With the expiration of the contract in question the going concern as an intangible value automatically became nil.

As an illustration, about three blocks away from the present garbage-reduction plant of the city of Philadelphia is an abandoned reduction plant which was put out of business by the contract for garbage disposal of said city having been obtained by the company controlling the present plant.

The city of Chicago can control by ordinance, when not by itself contracted for, the disposition of all the garbage.

The city then controls that which is of value and enables a profitable business to be done by the reduction plant during the 5-year term of contract between the Chicago Reduction Company and the city of Chicago. Hence why should the city pay, donate or present any additional sum to the Chicago Reduction Company for "going concern" value, when such is of its own creation and under its own control after Aug. 31, 1913?

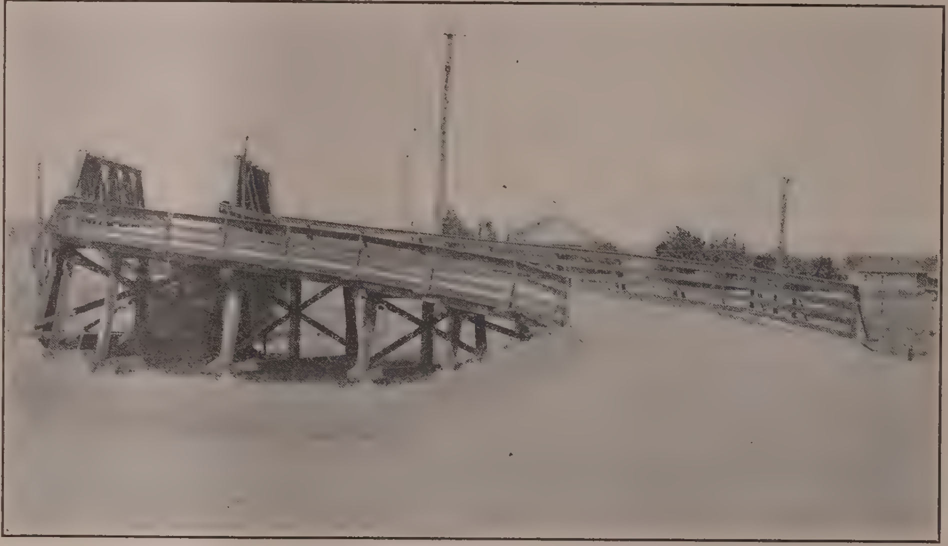
The city is entitled to value received. To cause it to pay an additional sum as going concern value would be similar in fact to its having granted a franchise to a corporation and then paying the corporation for its having done so.

The market value, or purchase price, in this case, is practically the going concern value. This value does not, therefore, depend upon the profits made during the contract of the Chicago Reduction Company with the city of Chicago or on the cost of reproduction with physical and functional depreciation deducted, but upon what the plant is worth to the city of Chicago as a willing purchaser.

The plant is worth to the city the value of the land, plus the salvage or scrap value, plus contract of appraisal obligations, plus an amount that may be termed the value of "temporary utility" or "availability for temporary use."

Dangerous Viaduct Approach

Non-co-operation of adjoining municipalities or of a county and city over joint structures on the boundary lines does not often lead to such monstrosities as the crooked viaduct approach on the outskirts of a little town in California, shown in the photograph on this page. That the view greets one on emerging from a beautiful new library building accentuates the incongruity of the bridge which spans the tracks of the Southern Pacific Company, the abrupt angle being made necessary on account of a jog in a city limit boundary line within which the approach had to be built.



California Viaduct Approach in Form of Letter S

Effect of Salts on Strength of Concrete

Tests to determine the effect of sodium chloride and calcium chloride, separately and together, upon 1:2:4 concrete were made at the University of Wisconsin during the winter and spring of 1912 by Messrs. H. E. Pulver and S. E. Johnson, instructors in mechanics. The test pieces were cured at temperatures of from 60 to 75 deg. Fahr. and below freezing. The results have been published in the "Wisconsin Engineer" by Mr. Pulver.

All mix was by volume and the salts were dissolved in the mixing water. The test pieces were 4-in. cubes, and those cured at a temperature below freezing were broken after fourteen and sixty days, while those cured at normal room temperatures were broken after fourteen, sixty and three hundred and sixty days. Four pieces were tested at each age for each batch of concrete. Atlas Portland cement was used, and to prevent variation in the quality it was mixed thoroughly in the beginning and a sufficient quantity stored in air-tight cans.

The test curves show that as the percentage of NaCl is increased there is a nearly straight-line decrease in the strength of the concrete cured under normal conditions. The effect of NaCl alone, when added to concrete cured at low temperature, is probably to reduce the freezing temperature, and hence retard the freezing of the concrete, thus permitting of its setting and hardening. The curves show an increase in strength for the addition of NaCl up to 12 per cent, after which there is a decrease. It may be that beyond 12 per cent the weakening of the concrete due to the excess of NaCl more than offsets the strengthening due to the reduction of the freezing temperature.

When $CaCl_2$ alone is added to the concrete, cured either at normal or low temperatures, the effect is to increase the strength up to about 4 per cent $CaCl_2$, at which point the maximum strength seems to be obtained. This increase in the strength of the concrete may be due to the acceleration of the setting of the cement by the $CaCl_2$. Serious disintegration was observed on the surfaces of the cubes cured at low temperatures with 6, 8 and 10 per cent of the $CaCl_2$. This disintegration did not appear on any of the cubes cured at normal temperature or where NaCl was used.

With concrete cured at low temperatures, the best effect seems to be obtained by using both NaCl and $CaCl_2$ in the mixing water. It was noted that a 2 per cent $CaCl_2$ and 9 per cent NaCl mixture appears to give the most satisfactory results. For concrete cured at low temperatures this mixture gives about as much strength as any of the mixtures tried, and for the concrete cured normally there was not a very great reduction in strength due to the addition of the salts in those percentages.

All tests were made with only one brand of cement. It is probable that there would be some variation in the results with other brands, but it is not thought that this variation would be great enough to affect the general conclusions. It is also possible that some brands of common salt might contain a sufficiently high percentage of calcium sulphate to affect the results to some extent.

The Metric System has been adopted in all countries except the United States, Russia and Great Britain, according to the annual report of the Decimal Association of England. "The Builder" voices the hope that the commendatory message sent the association by the Russian Minister of Commerce will help to induce Great Britain to accept the system.

Design of Steel Mill Buildings—Part I

Review and Discussion of Practical Considerations Governing Their General Features, Proportioning and Detailing

By R. Fleming, Designing Engineer, American Bridge Company, New York City

The purpose of this article is to call attention to some practical points that enter into the design of steel mill buildings. Articles written from the professor's chair are sometimes purely academic. The assumptions made, and upon which conclusions are based, are often found in actual practice to be incorrect. A thorough knowledge of the theoretical treatment of the various phases of mill building design given in the text books is necessary but will not in itself confer the ability to design a mill building that is economcal or adapted to practical needs. This knowledge should be supplemented by one's individual experience as well as by the experience of others.

The engineer having a building to design will usually consider the different features of his work in the order followed below.

Data

At the very outset he should possess himself of all available data. For what use is the building intended, and what are the probable uses to which it may be put? Upon these two questions largely depend the loads to be assumed for floors, the kind of covering for the roof, the amount of light, heat and ventilation required. The geographical location will decide the snow load. Sometimes the proposed building is within the limits of a city having a building code. A copy of this should be procured. Incidentally, a design based on sound engineering principles should be affected only in minor details by a building code, provided the code represents sound engineering principles. Local conditions may govern the selection of material to be furnished. As a matter of course, attention must be given to the preference of the owner.

If the building is to be used for manufacturing purposes, a lay-out of the machinery should be obtained from the mechanical engineer. This is necessary to locate interior columns to the best advantage. In these days of specialization we have also the heating and ventilating engineer, the illuminating engineer, the sanitary engineer, and others, engaged on the structure. The order of precedence may be as follows: Mechanical engineer, structural engineer, architect, other engineers.

When the steelwork is but a comparatively unimportant part of the structure the work of the architect precedes that of the structural engineer. When the steelwork predominates, the structural engineer should take the lead, while in the design of many mill buildings he is both engineer and architect.

It is seldom the good fortune of the engineer to be called upon to design an industrial plant from the start on a new site. Generally the proposed building is an addition to another building or a building added to a group of buildings. Most large manufacturing establishments are a growth from humble beginnings. Buildings are enlarged or new ones built to meet the exigencies of the time without much thought of future requirements. This growth will in most cases continue, and in designing new buildings provision should be made for their future extension. The conditions of the present work should be thoroughly understood.

In many cases data to the extent outlined can not be obtained. In work for export the inquiry is often very crude, little more than the uses to which the building is to be put and the approximate dimensions being given. Recourse must then be had to plants previously built for the same kind of work. A knowledge of buildings similar to those which the engineer is called upon to design is always valuable. The files of the technical papers for the past 10 to 20 years are rich in plans and descriptions which can be studied with profit.

Lighting

The demand of the industrial world to-day is for greater efficiency. To meet this demand in mill buildings attention should be given both to the machine and to the man. Better means of handling material will increase the output and lessen cost. Given a more favorable environment in the way of light, heat, and ventilation, the workman is more efficient, resulting in a larger and better output. In late years the question of proper lighting and its economic value has been widely discussed.

The United States Census of 1910 gives for the year 1909 the number of manufacturing establishments as 268,000, engaging 7,678,000 persons, of whom 6,615,000 were wage earners. The value of the products during the year was over twenty billion dollars, of which the portion added by manufacture (value of products less cost of materials) was over eight billion dollars. Assuming that the value of the output could be increased 1½ per cent, a low estimate, if better illuminating methods were used, we have an annual saving for all industries of $315,000,000.

Out of an average 24-hour day we have 11¾ hours of darkness, 4½ hours of cloudy or foggy atmosphere and only 7¾ hours of sunshine. Hence manufacturing plants must operate for part of the time by artificial light. We now have the illuminating engineer to deal with this feature.

The structural engineer, however, should, if it is possible, design his building to admit all the natural light needed during the day. This is generally done by using windows in the sides and, for the portion of the building the windows fail to light, skylights in the roof. The windows should be low, if there are benches along the wall or if light is needed near the sides, and they must be high to light the central portion of the room. A good plan is to have them begin 3 ft. from the floor and extend as high as possible. When balanced sash can be used, the expense of counterweights is saved.

A monitor, the purpose of which is to light or ventilate the building, is often built on the ridge of the roof, running from end to end, or stopping one panel from each end, with swing or sliding sash in the sides. The sash give ventilation as well as light. For the best results the monitor should have a width about one-fourth that of the roof on which it rests. Skylights in the roof are expensive, are liable to breakage, are a frequent cause of leaks, and when covered with snow do not admit light. Their use should be restricted to a minimum.

The proper proportion of glass surface to the side and roof areas of a building varies widely according to its uses. Fowler's "Specifications for Steel Roofs and Buildings" says as to this point:

"Windows shall be provided in the sides and clerestory or ventilator of a surface equal to not less than 10 per cent of the entire exterior surface of the building, in mill buildings, and of not less than 20 per cent in machine shops or similar buildings.

"At least half of the lighting specified shall preferably be in the form of skylights, of some form of construction, which shall entirely prohibit leaking."

This proportion is small for the sides and large for the roof. The East Side Machine Shop at Pencoyd, which is an unusually well lighted building, has one-eighth of the roof surface and one-half of the side and end surfaces covered with glass. Steel frame buildings having narrow concrete pilasters around the columns, and belt courses, also of concrete, at the floor and eave levels, with the balance of the sides filled with steel sash, are illustrated in the catalogues of the leading manufacturers of steel sash. With this construction almost the entire sides and ends are of glass. It might be noted here that the greater the expanse of glass the greater is the cost of heating and danger of breakage.

For large areas covered by one-story buildings, where a great amount of light is needed, what is known as "saw-tooth" roof construction has come into great favor. It is a series of trusses with one side vertical or slightly inclined and glazed, and the other side covered with roofing. Ketchum in his "Steel Mill Buildings" devotes a dozen pages to this style of roof construction. It should not seem necessary to admonish the designer that the glazed portion of such roof should face the north.

The question of heating is not within the province of this article. Suffice it to say that there are many systems of heating—a common one being the forcing of fresh air over heated pipes in a cast iron case and from thence distributed through conductors into various parts of the building. This system has the advantage of combing ventilation with heating.

Ventilation

The design of the building should provide for proper ventilation. The ventilating area required varies from ½ to 8 sq. ft. for every 100 sq. ft. of floor area, according to the uses of the building. When ventilation is the prime factor, little or no light being required, swinging shutters or fixed louvres, generally made of sheet metal, are used in the sides of the monitor in place of sash. Continuous rows of sash or shutters are sometimes placed in the sides of buildings, but more commonly ventilation from the sides is secured by the windows.

The steel sash people make their sash so that certain portions can swing on pivots. An operating device will enable the whole system of pivoted sash, whether in the sides of the monitor or in the sides of the building, to be easily controlled from the floor. A number of makes of tubular ventilators of galvanized steel are on the market. They give excellent service and can be placed on the roof where needed—the ridge being naturally the appropriate place. Ventilation is also secured artificially by forcing air through conductors by means of fans and blowers.

Important points to be considered in the choice of a roof covering are the uses to which the building is to be put, the roof pitch, the possibility of fire, the climatic conditions, the presence of gases, the appearance of the finished building, permanence and cost. The list of roof coverings is a long one; sheet metal, slate, tar and gravel on plank, tile, hyrib, tin, together with composition roofing of many kinds, are all familiar forms. Each has advantages for certain conditions.

Corrugated steel, either painted or galvanized, is a cheap covering and can easily be renewed when necessary. It is much used for sheds, car barns, rolling mills, and similar structures. It is laid on channel or angle purlins spaced to fit the sheets. The sheets most in use are 7, 8 and 9 ft. long. When laid each sheet should be supported at the center as well as at the ends by a purlin. The standard laps are 6 in. for the ends of sheets and one and one-half currugations for the sides. The best pitch for a corrugated steel roof is 6 in. to the foot. Five inches to the foot will answer, but when less than 4 in. the roof is liable to leak during a shower accompanied by a high wind, the water being pushed up between the end laps of the covering. A greater lap than 6 in. will partly remedy this trouble but will not do away with it altogether.

Preventing Condensation

When the temperature of the room inside is considerably higher than that of the air outside, condensation will take place on the under side of the corrugated steel and drip on the floor below. In such cases other covering should be used or sheathing placed under the corrugated steel. An "anti-condensation" lining under the sheets is sometimes used; it is made of one or two layers of asbestos paper and one or two layers of tar paper, resting upon galvanized wire netting attached to one eave purlin and stretched over the ridge to the other eave purlin.

In buildings for export to warm climate there is a feeling against the use of corrugated steel. In a tropical climate its deterioration is rapid. This may be retarded by galvanizing but can not be prevented entirely. It is much the same when subject to the action of certain gases. For this reason corrugated iron may be called for, although it is more expensive and often difficult to procure. In a copper refining plant near Bayonne, N. J., the corrugated steel roof and sides of a building had to be renewed 18 months after it was finished. Another building near by, erected at the same time and subject to the same uses, with roof and siding of corrugated iron, is still doing good service after eight years of use.

Slate laid on plank is a favorite covering. Its first cost is high, but it is lasting, is not affected by gases, and seldom needs repairs. A normal pitch is 7 in. to the foot, though 6 in. is frequently used. For low pitches—4 in. or less to the foot—the same trouble with leaks as with corrugated steel will be experienced, but it will be intensified by the greater number of laps. Slaters' cement for the laps is a salve, not a cure.

Tar and gravel on plank sheathing have been used for many years when the pitch is from ½ to 2 in. to the foot. For a greater pitch than this the covering will wash off during a hard rain with the result that the roof will leak.

The fire-resisting qualities of reinforced concrete are causing it to come more and more into favor. It can be used for any pitch and adapted to any waterproof covering.

It is highly important that the choice of covering be adapted to the pitch of the roof or vice versa. The lack of this adaptation is a prolific cause of leaky roofs, and a leaky roof not only entails expense for repairs but, what is sometimes more serious, damage to stock or machinery.

Sides

The siding for mill buildings varies, though not through as wide a range as the roofing. If corrugated steel is used for the roof, the sides are often made of the same material, though of lighter weight. If No. 20 gage is used for the roof, No. 22 can be used for the sides. Less than No. 22 should not be used for roofs nor less than No. 24 for siding.

To the common construction of brick walls between steel columns but little objection can be raised. There should be steel lintels over window and door openings unless the openings are narrow, when brick arches will answer. If the sides are thin curtain walls, the lintels should extend from column to column and be designed to carry the wall above. They thus act as struts between the columns as well as beams. In the buildings of the Ambridge plant of the American Bridge Company, the side walls throughout are of concrete 8 to 12 in. thick. The walls are carried by the steel columns.

No one kind of floor is adapted for universal use. For rolling mills, forge shops or foundries, which have only a ground floor, earth well tamped will generally answer. Sometimes the ground floor is made of brick or cement. For the floors of machine shops, mills and factories a common practice is to use yellow pine planks with a wearing surface of 1 in. hard maple. Buckle plates covered with concrete with a cement finish, steel trough floors, corrugated steel arches and concrete, checkered steel plates, reinforced concrete, and a number of patented floors are all in more or less use, though mostly for upper floors.

Roof Loads

The loads for which the steel portion of a roof is ordinarily designed are the weight of the structure itself, called the dead load, the snow load and the wind load. The dead load includes the weight of the steel truss, the purlins, roof covering, bracing, and any other loads which are permanently a part of the roof. This load varies from 5 or 6 lb. per square foot for the lightest kind of corrugated steel roofs to 30 or 40 lb. per square foot for concrete roofs. The snow load varies according to locality. Fresh fallen snow weighs from 5 to 12 lb. per cubic foot. A load of 20 lb. per square foot is ample provision for snow load on roofs in the latitude of New York.

The wind load may be assumed equivalent to a pressure of 20 lb. per square foot. It is good practice, though not universally adopted, to assume for roof trusses a combined dead, snow, and wind load per square foot of exposed surface, applied vertically. Schneider's "General Specifications for Structural Work of Buildings" follows this method and recommends that for ordinary roofs up to 80-ft. span, in climates corresponding to that of New York, the following loads per square foot be used:

	Lb.
Gravel or composition roofing on boards, flat slope, 1 to 6, or less	50
Gravel or composition roofing on boards, steep slope, more than 1 to 6	45
Gravel or composition roofing on 3-in. flat tile or cinder concrete	60
Corrugated sheeting on boards or purlins	40
Slate on boards or purlins	50
Slate on 3-in. flat tile or cinder concrete	65
Tile on steel purlins	55
Glass	45

In tropical climates, where there is no snow, the above loads may be reduced 10 lb. In some parts of New England and the Northwest where there are unusually heavy snows the loads should be increased. The reader acquainted with the text books and taught that the wind acts normal to the roof surface and that the stresses in members differ according to their location on the windward or leeward side will at once question the right to consider the wind acting vertically along with the dead and snow loads. The answer is that the error, if there be any, is negligible.

It is needless to make a separate calculation for each kind of load and each case of wind loading. The reader can easily prove this for himself. Assume a wall-bearing roof truss of the Fink type, say of 60-ft. span, a pitch of 6 in. to the foot, and trusses 16 ft. apart. Of the 40-lb. per square foot load, let 20 lb. be the wind load and 20 lb. the dead and snow load. Using the graphical method proceed as follows:

Graphical Method

(1) Make a stress diagram for the 20-lb. dead and snow load, the loads being concentrated at panel points of the top chord and acting vertically;

(2) Make a stress diagram for a wind pressure of 20 lb. on one side and normal to the surface, the loads being concentrated at the top chord panel points of one side and the truss considered to have both ends fixed;

(3) Make a stress diagram for the same wind pressure, considering the end on the truss on the windward side to be fixed and the end of the leeward side to be free or on rollers;

(4) Make a stress diagram for the same wind pressure, considering the windward side of the truss to have a roller end and the leeward side a fixed end;

(5) Show on a diagram of the truss the stresses due to a vertical load of 40 lb. per square foot of exposed surface. These stresses are double those found in (1).

By a comparison of the algebraic sum of stresses found under (1) and either (2), (3) or (4) with (5) it will readily be seen that the differences do not affect the size of a single member.

Shafting is sometimes carried by the lower chords of the roof trusses. This causes stresses indeterminate in amount, but if the shafting is light a stiff lower chord made of two channels or plates and angles will provide for it. If the shafting is heavy the whole truss should be strengthened by adding 10 lb. per square foot to the assumed roof load and using a stiff lower chord. Sometimes a given load is to be lifted from any point of the lower chord, or it is to carry a monorail or light crane. These loads are all definite in amount and stresses due to them can be easily calculated.

Floor Loads

The dead load of the floor, consisting of the weight of the steel supports and the materials composing the floor proper, varies between wide limits—from 20 lb. per square foot to 80 lb. and more, according to the floor construction. The live load which should be used varies, according to the uses of the room, from 60 lb. per square foot for the floors of pattern and template shops to 600 or even 1000 lb. per square foot for the charging floors of cupola rooms. Machine shop floors, owing to vibration of machines and the liability of concentrated loads, should be calculated to carry from 150 lb. per square foot when the machinery is light to 300 lb. per square foot when it is heavy.

For floors of factory buildings, with machines as close together as they can be placed, the live load should seldom be assumed less than 200 lb. per square foot. For heavy work it may be more. The weights of particularly heavy machines should be determined, and ample provision made for carrying them. Note should be made if stock or materials are to be heaped upon the floor in any considerable quantity. In speaking of floor loads the nontechnical man generally means the live load and not the combined live and dead. There should be no misunderstanding with the owner on this point.

Crane and Special Loads

The traveling crane is an important feature of the modern mill building. Hand power cranes are occasionally used, but the vast majority of traveling cranes are electrically driven. Traveling cranes vary in their lifting capacity from 3 to 5 tons, used in light machine shops, to 150 and 175 tons, used in locomotive shops and steel-furnace buildings. Probably more cranes of 10-ton than of any other capacity are used. Up to about 60 tons capacity they are generally carried on a two-wheel truck at each end with a wheel base of 8 to 12 feet.

Cranes of 60 to 120 tons capacity should be carried on a four-wheel truck at each end, while the trucks for 150-ton cranes should have eight wheels. It is difficult to procure rails or design girders that will properly withstand the great concentrated loads that come from too few wheels. For this reason preference should be given to the crane having the greater number of wheels under the trucks. The maximum loads that can come upon the wheels should be obtained from the crane maker. If the make of crane has not been decided upon, the loads given in published tables may be used. The wheel bases and maximum wheel loads of standard cranes, with the exception of those of one maker, vary but little. It is the custom to add 25 per cent. to the maximum wheel loads for impact, as the crane is a moving load and is liable to be started quickly or brought to a sudden stop.

In addition to the vertical loads the girders or runway of a traveling crane should be designed for lateral stiffness. The neglect of this has been the cause of failure of more than one crane runway. The lateral load may be assumed to be 20 per cent. of the transverse moving load—one-half at each end of the crane divided equally between the truck wheels. This assumes that double flange wheels are used on one side. The owner may request that a crane runway be designed to carry a heavy load for a given distance from one end and a lighter load for the remaining portion. A design of this kind should not be drawn, for, either through ignorance or intention, the heavy load will sometimes be brought on the portion designed for the light load. The blame for any disaster invariably falls upon the designer.

Jib cranes are attached to roof trusses, to floor beams, and to columns. The horizontal force in any position of the boom should be analyzed. In a power plant the question of coal storage should be taken into consideration and bunkers or bins designed where required. Heavy economizers may be carried on the floor of the boiler room, and dynamos on the floor of the engine room. Many buildings carry special loads peculiar to the trade for which they are built—in cement mills are storage bins and in smelting plants are ore bins. The character of loading should be obtained.

Unit Stresses

With the external loads given, the internal stresses in the supporting members of a structure can generally be determined. Some members have direct stresses, others bending stresses, and others a combination of both. In proportioning members the following may be assumed as unit or working stresses in pounds per square inch for open hearth medium steel generally used for mill buildings:

Tension, net section	16,000
Bending on extreme fibers of rolled shapes and riveted girders	16,000
Shear on plate girder webs (net section)	10,000
Shear on rivets (shop)	12,000
Shear on rivets (field) and bolts	10,000
Bearing pressure on rivets (shop)	24,000
Bearing pressure on rivets (field) and bolts	20,000

For steel members in compression there are a number of column formulæ in common use. An article in the Engineering Record of July 12, 1913, page 41, gives twenty-seven such formulæ in use in this country and Europe. The straight line formula, $S = 16{,}000 - 70\, l/r$, is recommended, in which S is the allowable stress per square inch, l, the effective length of member in inches, and r, least radius of gyration of the member in inches. The maximum value of S is 14,000.

Wind Pressure

It is with considerable hesitation that the writer approaches this much-discussed subject. Much that is written on wind pressure and the stresses resulting therefrom is based on assumptions that are too uncertain to warrant the accuracy and amount of work involved in their treatment. There are many steel mill buildings that have been standing for the last 15 or 20 years in good condition, many members of which, if calculated for wind according to some of the theories advanced, would be stressed beyond their ultimate strength. This is especially true in the case of side girts supporting metal covering. If they are considered as beams, the calculated stresses for bending are far beyond allowable limits; if considered as suspension rods between columns the connections are entirely inadequate.

In an ordinary mill building the wind does not play the important part that it does in a high office building, but the designer should know as far as can be ascertained to just what extent it affects the structure and make assumptions accordingly. The wind as a part of the roof load has been mentioned under "Roof Loads." We have yet to dispose of the problem of wind on the sides of a building.

The general method is to consider the wind stresses carried to the foundations transversely by each separate bent of the building. The Chicago building law, which is one of the latest and best of our building codes, has the provision: "All buildings shall be designed to resist a horizontal wind pressure of 20 lb. per square foot for every square foot of exposed surfaces. For stresses produced by wind forces combined with those from live and dead loads, the unit stress may be increased 50 per cent over those given above; but the section shall not be less than required if wind forces be neglected."

With the wind blowing horizontally, a load of 20 lb. per square foot on the side of the building and on the vertical projection of the roof surface is thought ample. The ends of the columns are considered fixed at the base, with the point of contraflexture midway between the base and the foot of the kneebrace, or, in the absence of kneebraces, midway between the base and the gusset plate connecting the truss to the column. One-half the wind load on the surface between the foot of the kneebrace or gusset plate and the column base will be carried directly by the column to the foundation and need not be considered further. The remaining side pressure is concentrated at the foot of the kneebrace or gusset plate and the panel points of the roof truss. The bases of the columns are each assumed to take one-half of the horizontal reaction. A stress diagram should be made based on these assumptions.

The stresses due to the dead and snow load can easily be found either by taking a proportion of the stresses already found due to the combined dead, snow, and wind load on the roof or by making a new stress diagram. Comparing the two sets of stresses in accordance with the Chicago building law it will readily be seen what unit stresses are to be used in proportioning the members. In no case, however, should the sectional area of any member be less than that required by the combined dead snow, and wind load mentioned under "Roof Loads" using the working stresses given under "Unit Stresses." In addition to direct stresses, the columns take bending from the wind load. The maximum bending occurs in the leeward column at the foot of the kneebrace and at the column base, being equal to one-half the distance from the foot of the kneebrace or the gusset plate to the ground, multiplied by the horizontal reaction at the point of contraflexure. From the bending moment the flange stresses in the columns are found. As these flange stresses are due to wind, the higher unit stress can be used in determining the flange area required. Making comparisons in the manner mentioned above it can be determined whether the sectional area of the column required by the direct stress should be increased.

A week point in the above method is in the assumption of the point of contraflexure. In actual practice the bases of columns are seldom fixed sufficiently to warrant such an assumption. Again the assumption that the horizontal reaction is equally divided between the two columns may or may not be true. A compensation for these errors is found in the fact that a wind load of 20 lb. per square foot for areas near the ground is excessive.

Another method is to consider the columns as beams uniformly loaded and the reactions at their tops taken, by means of a system of bracing in the plane of the lower chords of the roof trusses, to the ends of the building and thence to the ground by diagonal bracing in the plane of the ends. This method is well adapted for buildings where the bottom chord is a considerable distance above the floor, provided the building is not so long that it requires expansion joints and that there are no openings in the ends to interfere with diagonal bracing.

The second and concluding part of this article, taking up particularly the proportioning of members and detailing, will appear in the next issue.

Motor Trucks for Rescue Work

A thoroughly equipped motor hospital recently completed for the Bureau of Mines for use in Pittsburgh discloses a new use for the auto truck, a variation of which might find an application in the work on water-supply projects such as the Catskill and Los Angeles aqueducts. The first vehicle of the new type includes in its equipment two pulmotors, six oxygen tanks, fifty regenerators, an oxygen pump, two stretchers, a life line, a box of explosives, lanterns, tents, fire extinguishers, a complete telephone system and a full complement of hose, axes, saws, miners' tools and medical and surgical supplies. Even the bird and cage used in detecting poisonous gases in mine shafts are to be found in the ambulance.

The truck was demonstrated at the recent convention of the American Mine Safety Association in Pittsburgh. The new car was built according to Government specifications by the White Company, of Cleveland, Ohio.

Sinking Piers through Soft Clay

The 166 x 324-ft. twenty-one-story office building for the Continental and Commercial National Bank, at Quincy and Adams Streets in Chicago, occupies the site formerly covered by several brick buildings and the eleven-story steel-frame Rand & McNally building, all of which were torn down to make room for the present structure.

RAZING OLD BUILDINGS

The sidewalks around the site were first protected by a bridge of the ordinary type, 10 ft. wide, with a capacity of 300 lb. per square foot, and operations were commenced by the removal of the three smallest buildings, from four to seven stories high, of ordinary construction, with wooden interior columns and wall-bearing wooden floorbeams. The dismantled brickwork and the rubbish were delivered through separate chutes to wagons on the first floor. Three ten-story buildings with wooden floors, steel I-beams and cast-iron columns were wrecked in the same way, except that a derrick was provided to lower the I-beams on trucks in the street, the chutes being arranged zigzag to reduce the velocity of the materials.

The Rand & McNally building, which was one of the earliest of the steel-frame office buildings in Chicago, had steel columns in one-story sections supported on grillage foundations composed of several tiers of rails embedded in concrete. The glass and marble work, doors and sash, were first removed and a stiffleg derrick was installed on the roof to handle the steelwork. Six chutes were set up for bricks and rubbish, and after the bolts and rivets were removed or cut off the columns and connected header beams were pulled down in groups by the derrick and lowered by two ginpoles to wagons in the center court. A 20-ton platform scale was installed and on it all of the steelwork in the different buildings, about 5500 tons, was weighed out.

Two stiffleg derricks were set up on the cellar floor and with them 8600-lb. cast-iron balls were raised 90 ft. and dropped on the old foundations to break up the heavy concrete piers, some of them 2 ft. thick, with five tiers of rails embedded in them. The rails were gripped by tongs and pulled out by the derrick tackles.

The work was accomplished by a force of about one hundred men in each of the day and night shifts, and required about seventy-eight days for the Rand & McNally building and forty-four days for the other buildings. All of the steel, some of which had been in use about twenty-five years, was found in good condition, with little evidence of rust or of electrolysis. although numerous motors had been used in the building. The salvage on the materials recovered from the wreck paid for the cost of razing the building.

EXCAVATING CELLAR AND PIER WELLS

As soon as the removal of the first buildings afforded sufficient space for it a 1-yd. steam shovel started the cellar excavation, going down 10 ft. below the old floor to a depth of about 16 ft. below the curb to new subgrade, and delivered to about fifty two-horse wagons working day and night and hauling the spoil over inclined planes to the street level, thence carrying it about seven blocks to the dump scow. The total excavation amounted to about 60,000 yd. of fill and clay. After it was completed about 400 foundation pits, 7½ ft. in diameter, were sunk to rock at a depth of about 85 ft. below the curb.

They were excavated in the usual Chicago manner by the use of circular sheeting, consisting of 2 x 6-in. wooden staves 8 ft. long, set outside of 2¼ x ¾-in. flanged iron rings 4 ft. apart vertically. The soft clay was removed inside of these cylinders by two men with sharp spades and grub hoes, assisted by two men on the surface of the ground, who operated capstan heads commanding the hoist lines for the ½-yd. buckets. From twelve to sixteen wells in two rows were operated by a single hoisting engine running a rope drive which engaged all of the capstan heads and operated them continuously, so that it was only necessary to take turns of the hoisting rope around it and tail off the slack in order to raise the bucket at any time. Little pumping was required, the wells being sunk at an average rate of 4 ft. in each of the three eight-hour shifts.

Rope Drives Operating Hoists for Spoil Excavated from Foundation Wells

The regular level surface of the rock was cleaned off, the iron rings removed, and the wooden staves left in position when the wells were filled with 1:2:4 concrete made of Universal Portland cement and 1½-in. stone, delivered from a mixer to six 1-yd. side-dump steel cars on 2-in. service tracks. The concrete was mixed soft and deposited in the wells through 10-in. steel chutes. After 10 ft. of concrete were deposited men entered the pits and examined and leveled it, another course was deposited, and so on, until the pit was filled, which was accomplished in about four hours. The top of the concrete was carefully leveled about 1½ in. below the required elevation for the bottom of the grillage, and two 2 x ½-in. oak strips about 4 ft. long were carefully set and leveled on it to receive the first tier of 12, 18, or 24-in. grillage beams, which were grouted and boxed and concrete deposited.

The foundation work was executed by an average force of about six hundred men under the direction of Mr. A. H. Petersen, superintendent for the Thompson-Starrett Company, general contractors. D. H. Burnham & Company are the architects. The American Bridge Company was the contractor for the fabrication and Bergendahl & Brother for the erection of the structural steel. The old buildings were wrecked by Mr. F. Krug.

Multiple-Entry Method Proposed for Rogers Pass Tunnel

Work was recently started on the 5-mile, double-track tunnel for the Canadian Pacific Railroad under Rogers Pass, B. C., and, although the exigencies almost sure to arise in a work of this magnitude may require some radical change in plan, the present scheme is to drive parallel to the main bore and a little to the north a tunnel 7 x 8 ft. in section from which cross-cuts into the main line will be made to permit of carrying on the work at a number of points simultaneously.

The "pioneer" tunnel, as the auxiliary bore is called, will be equal in length to the main tunnel and will have a double line of tracks over which the material from the inner headings of the main line will be hauled out by electric motors. For the greater portion of its length the main bore will be driven through at the outset with a cross-section not much larger than that of the pioneer tunnel, and later on, in addition to the boring from either end, which will be full size from the beginning, machinery will be installed at two or more interior points, and the working enlarged to the standard size of 22 x 30 ft. required by the contract. The pioneer tunnel will be driven from both ends, and it is thought probable that the first cross-cut will be made about 2000 ft. in from each portal.

Messrs. Foley, Welch & Stewart, of Vancouver, B. C., are the contractors for the tunnel, and Mr. A. C. Dennis is superintendent of construction at Glacier, B. C.

OILING MACADAM-PAVED STREETS in Madison, Wis., during the summer of 1912 cost the city $5,283.21, according to the annual report of the city engineer. Of this amount $3,874.52 was for the cost of the oil and the remainder, $1,408.79, was for all the other items chargeable to the work. The total mileage was 12.1 having an area of 192,000 sq. yd. The unit cost was about 2.75 cents per square yard. The oiling the past season was done mainly on streets which received oil the previous season and which needed another application on account of heavy traffic.

Concrete Roads

Abstract of Discussion by A. N. Johnson, State Highway Engineer, Illinois, before the Third American Road Congress at Detroit

It is evident that if one portion of a concrete road surface exhibits a special weakness it must be due to non-uniformity of the concrete. This will happen if soft pieces of material are in the aggregate. Many gravel banks contain enough soft pebbles to render the gravel useless for concrete-road work, though frequently these soft pebbles have somewhat the general appearance of the sound material and, unless a careful examination is made, will not be detected.

Another cause for local weakness in concrete is the non-uniform distribution of the mortar and the aggregate. It frequently happens, particularly with very wet mixtures, that the coarser aggregate will not be thoroughly distributed but will occasionally be separated into small collections or nests of the larger pieces, which contain a very small amount of mortar. There is no mixer that has come under the writer's observation which will mix and deposit a batch so uniformly that all portions will contain the proper amount of coarse aggregate and mortar. It is necessary, therefore, in the construction of a concrete road to have one or two men constantly at hand to correct such uneven distribution by shoveling out the pockets of coarser aggregate to be found in the center of nearly every batch that is deposited on the road and replacing these pockets with mortar, which usually runs to the bottom and edges of the batch.

Cracks

The causes of the other three defects noted by Mr. Rogers (see Engineering Record, Oct. 11, 1913, page 409)—longitudinal cracks, transverse cracks and diagonal cracks—are perhaps more difficult to ascertain in a given instance. The data presented have been rearranged according to the age of the various pieces of road in the table following. In order to have a proper basis of comparison the number of cracks per mile has been computed for the roads built in 1909, 1910, 1911 and 1912 respectively.

Cracks per Mile for Each Year

	1912	1911	1910	1909
Transverse	7.4	9.9	17.3	26.2
Longitudinal	9.8	29.8	64.8	70.0
Diagonal	1.9	4.5	11.7	2.3
Totals	19.1	44.2	93.8	98.5

At once one is struck with the increase in the number of cracks with each succeeding year. This increase is particularly marked during the first three years, but apparently falls off sharply during the fourth year.

A notable coincidence is that the curve showing the total number of cracks per mile of all kinds is approximately a straight line for the first three years, or, in other words, the increase in the number of cracks was approximately the same for each of these years.

There is also considerable agreement of the curves for the longitudinal, transverse and diagonal cracks during the first three years, but no such agreement for the fourth year. It is to be borne in mind, however, that for the fourth year data are given for only two pieces of road about 1.3 miles in length. However, as these pieces were laid with a leaner mixture than much of the other work and constitute the first concrete work done by the Wayne County Commissioners, all the evidence would point to the conclusion that this stretch of road would show at least as many defects as will appear later in the other roads.

The value of future observations to be made at intervals of not less than three months will be evident. These observations should note carefully all the cracks that are found, both as to their character and, particularly, as to their distribution, so that in the future the location of new cracks as they appear may be known, as well as the approximate time of the year when they occur.

Probable Causes of Cracking

It can be fairly assumed that the slabs of concrete are approximately of the same strength, offering practically the same resistance to all exterior forces to which they are subjected.

Among the forces which would cause the slabs to crack are variations in temperature, extraordinarily heavy traffic loads and unequal settlement of the foundations. It is more than probable that the cracks are due to no one of these causes alone but rather to their accidental combination. If the changes in temperature were the prime cause, it could be expected that all slabs would show approximately the same number of cracks, for the reason that all portions of the road are subjected approximately to the same temperature variations. But an examination of the roads shows many slabs in continuous stretches which have not cracked at all, regardless of the age of the road. It can, therefore, be concluded that temperature changes alone are not sufficient to cause cracking of the slabs in general.

While extraordinarily heavy moving loads may not be so generally applied to all the slabs, and therefore not cause as uniform an appearance of cracks, yet if this be a principal cause it could be expected there would be a large number of contiguous cracked slabs. But there is practically no evidence to indicate that there have been loads heavy enough to cause the pavements to crack from this reason alone.

The third cause that has been mentioned—unequal settlement of the foundation—is one that would not be generally distributed; for while settlement might occur in one place, there very likely would be considerable distances where it did not occur. Moreover, it will be appreciated that the manifold conditions which would produce unequal settlement would cause an erratic occurrence of any results from this source. And the haphazard occurrence of the cracks would indicate that the cause is not general throughout the length of the roads. The writer is, therefore, of the opinion that the majority of the cracks are due primarily to unequal settlement of the foundation or subsoil beneath the broken slabs. It is quite possible that settlement by itself would perhaps not be sufficient to cause the slab to crack, but on the passage of heavy traffic or with the movement of the slab under temperature changes there would be added just enough extra stress to overcome the strength of the slab and produce the crack. If, therefore, it is possible to guard against unequal settlement of the foundation, one of the chief sources of cracks in the pavement will be overcome.

The writer has observed that the concrete slabs usually do not crack until after they have been in service about a year. The first cracks generally appear during the late spring and early summer following construction. An explanation is that during the winter and early spring the ground is well saturated with moisture and dries unevenly; the consequent settlement is therefore not uniform. Special precaution with the underdrainage should accordingly be taken to provide thorough and uniform draining.

It is probable, as further studies are made, that the causes producing transverse cracks will be found to be somewhat different from those producing longitudinal cracks. One special cause of the latter is to be found whenever a concrete road is built over a hard road bed, the concrete slab being somewhat wider than the compacted center of the old road. The soil at the sides, which is not so compact as the center, will settle more, causing the concrete slab to break longitudinally with the road. To prevent this it is recommended that the old roadbed should be thoroughly loosened and then rerolled with the sides.

The writer does not believe that it is necessary to place expansion joints as close together as 25 ft. On three pieces of work constructed under his supervision in 1912, totaling 2.3 miles, none of the sections was less than 50 ft. in length, many being 60 and 75 ft., and one or two as long as 100 ft. The number of cracks per mile in these roads at present is: Transverse cracks, 9.1; longitudinal cracks, 9.1, or 18.2 cracks per mile. The total number of cracks in the 1912 work on the Wayne County roads, using 25-ft. sections, is 19.1. It will be necessary to have much more data before it can be definitely concluded as to how frequent the joints should be; but from the observations that have been made it seems that the joints may be made further apart than 50 ft. rather than nearer together.

A concrete proportion 1:2:3½ is amply rich for road service. As the writer pointed out last year at the American Road Congress, if all the aggregate is to be well surrounded with mortar (assuming 50 per cent voids), the sand should be in excess of one-half the volume of the coarse aggregate. Actual service has demonstrated the value of using this mix rather than 1:1½:3 or 1:2:4.

Lest a wrong impression should be gained from the emphasis laid upon the defects found in the Wayne County roads, the writer would state that from a number of personal inspections he feels that the Wayne County Commissioners have amply justified their adoption of this form of construction.

Rules Governing the Use of Heavy Vehicles on Highways

In order to protect effectively the many miles of improved highways being built within the State, the New York Highway Commission has promulgated rules governing the use of heavy vehicles on the roads. The regulations became effective Oct. 20. They are as follows:

Section 1.—No traction engine, road engine, hauling engine, trailer, steam roller, automobile truck, motor or other power vehicle shall be operated upon or over the State or county highways of this State, the face of the wheels of which are fitted with flanges, ribs, clamps, cleats, lugs or spikes. This regulation applies to all rings or flanges upon guiding or steering wheels on any such vehicle. In cases of traction engines, road engines or hauling engines which are equipped or provided with flanges, ribs, clamps, cleats, rings or lugs, such vehicles shall be permitted to pass over said highways provided that cleats are fastened upon all the wheels of such vehicles, not less than 2½ in. wide and not more than 1½ in. high, so placed that not less than two cleats on each wheel shall touch the ground at all times, and that the weight shall be the same on all parts of said cleats.

Section 2.—No traction engine, trailer, steam roller, automobile truck, motor or other power vehicle shall be operated upon or over the State or county highways of this State, nor shall any object be moved over or upon any such highways upon wheels, rollers or otherwise, in excess of a total weight of 14 tons, including the vehicle, object or contrivance and load, without first obtaining the permission of the State Commission of Highways as hereinafter provided. No weight in excess of 8 tons shall be carried on any one axle of any such vehicle.

Section 3.—The tire of each wheel of a traction engine, road engine, hauling engine, trailer, steam roller, automobile truck, motor or other power vehicle (except traction engines, road engines, and hauling engines) shall be smooth, and the weight of such vehicle, including load, shall not exceed 800 lb. upon any inch in width of the tire, wheel, roller or other object, and any weight in excess of 800 lb. upon an inch of tire is prohibited unless permission is obtained from the State Commission of Highways as hereinafter provided.

Section 4.—No motor or other power vehicle shall be operated upon any State or county highway of a greater width than 90 in., except traction engines, which may have a width of 100 in.

Section 5.—No traction engine, road engine, hauling engine, trailer, steam roller, automobile truck, motor, or other power vehicle, carrying a weight in excess of 4 tons, including the vehicle, shall be operated upon any State or county highway of this State at a speed greater than 15 miles an hour; and no such vehicle carrying a weight in excess of 6 tons, including the vehicle, shall be operated upon any such highway at a speed greater than 6 miles an hour when such vehicle is equipped with iron or steel tires, nor greater than 12 miles an hour when the vehicle is equipped with tires of hard rubber or other similar substance.

Section 6.—The State Commission of Highways of the State of New York, upon proper application in writing, may grant permission for the moving of heavy vehicles, loads, objects, or structures in excess of a total weight of 14 tons over its State and county highways upon proper application in writing being made therefor, and under such restrictions as said commission may prescribe.

Section 7.—The owner, driver, operator or mover of any vehicle over any State or county highway shall be responsible for all damage which said highway may sustain as a result of a violation of any of the provisions of the foregoing rules and regulations, and the amount thereof may be recovered in an action of tort by the State Commission of Highways or by any county superintendent of highways of any county or by any town superintendent of highways of any town in which said violation occurs.

Section 24 of Chapter 25 of the Consolidated Laws, entitled "The Highway Law," provides that any disobedience of any of the foregoing rules and regulations shall be punishable by a fine of not less than $10 and not more than $100, to be prosecuted by the town, county or district superintendent, and paid to the county treasurer to the credit of the fund for the maintenance of such highways in the town where such fine is collected.

IRRIGATION OF THE ADANA DESERT in Arabia is to be done by the Turkish Government. A contract has been signed with a German contractor for the construction of the works. It comprises an area of 1,235,000 acres.

Greater Vancouver Joint Sewerage Scheme

Organizing and Financing Project for Draining 55,600 Acres, with Data on Rainfall and Float Tests of Dispersion at Outfalls

The very rapid increase of population in and around Vancouver, B. C., requiring a corresponding development of sewerage systems, has brought about the necessity for more or less sewer construction in the near future in each of the natural drainage basins of Burrard peninsula. This peninsula includes 55,600 acres, of which only 9500 acres are within the present boundaries of Vancouver, the remainder being in New Westminster, South Vancouver, Point Grey and Burnaby. However, the ultimate necessity of a joint sewer system embracing the entire peninsula was early recognized, and in 1910 steps were taken to secure united action before there should be constructed by any of the independent municipalities, sewer systems which might not harmonize with the scheme most desirable for all.

Accordingly, in April, 1911, the several municipalities, with the exception of New Westminster, appointed a committee, composed of two representatives from each council, for the purpose of inquiring into the best means of placing the question of sewerage and sewage disposal on a sound and permanent basis. This committee retained Mr. R. S. Lea, consulting engineer, of Montreal, to advise them as to a general scheme for the whole peninsula and the best means of constructing and maintaining such a scheme in a state of efficiency. Acting on these instructions, a comprehensive report was submitted early this year and adopted by the joint committee.

The report advised the formation of a permanent sewerage board, and in February, 1913, there was drafted an act, based on the recommendations of the report, forming such a board. The Provincial Government was then approached with a request for legislation, and at the same time was asked for financial assistance in the form of a Government guarantee for a bond issue. With some minor amendments, the act passed the legislature and became effective Aug. 1, 1913.

This act virtually gave Government sanction to the board of control, a Government guarantee to secure the bonds which would furnish money for construction and empowered the board to construct and maintain sewer systems in accordance, substantially, with the report which was submitted by Mr. Lea. The act provides that the chairman of the board shall be appointed by the Lieutenant-Governor in council, the other members also being so appointed on recommendation of the municipalities which they represent. The chairman and other members of the board "remain in office during the pleasure of the Lieutenant-Governor in council," who fixes the salaries of each.

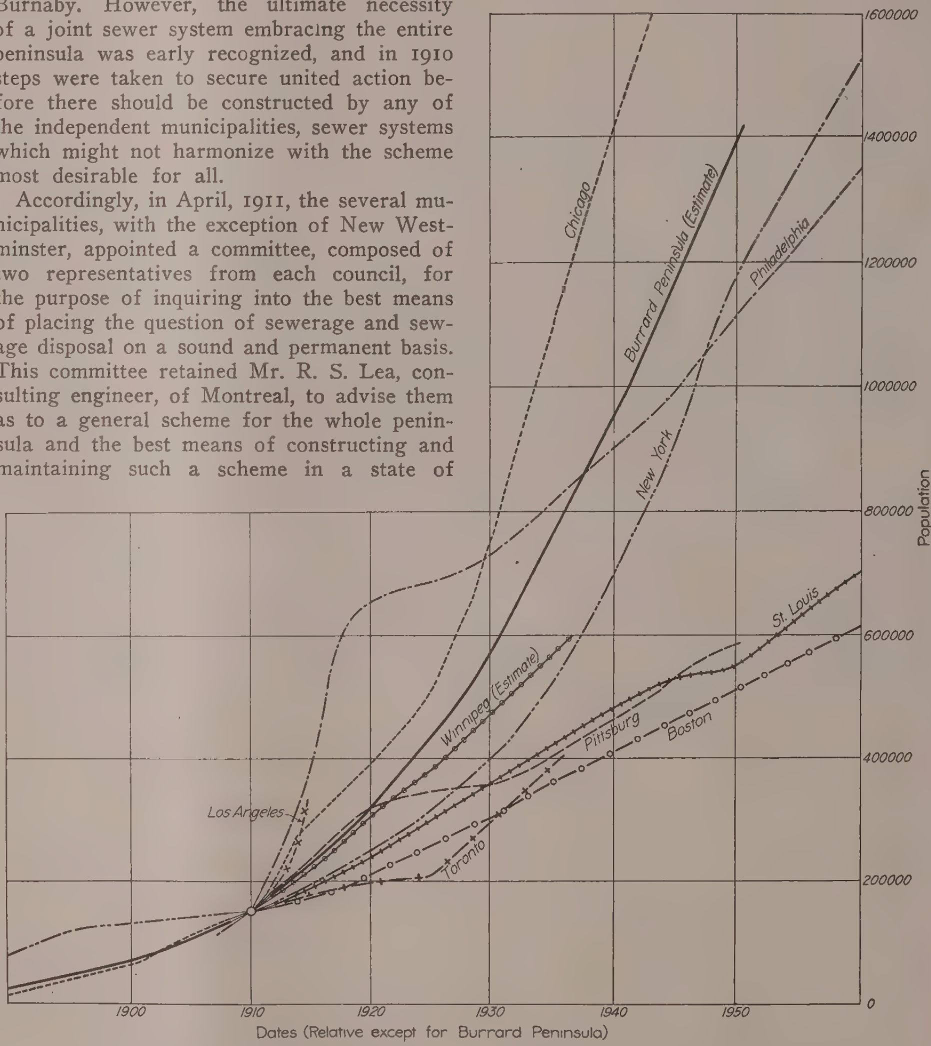


Chart Used in Estimating Increase of Population of Burrard Peninsula

The Government guarantee covers the interest on the $10,000,000 for which the board is empowered to issue bonds and $5,000,000 of the principal. Anticipating the possibility of some time being required to dispose of the bonds, the board has been authorized to borrow $350,000, so that there shall be no delay in carrying out that part of the work required at once.

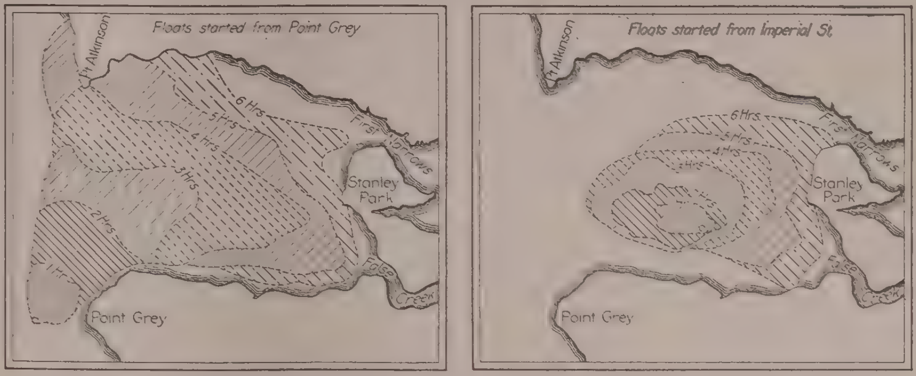
It was proposed to finance the sewer development on forty-year bonds and to design the systems so as to be of adequate size for handling the sewage of the probable population in 1950. In order not to burden the rate payers of to-day with an excessive capital outlay on sewers for future needs, estimates embodied in the report were divided into "immediate construction" and "deferred construction"—the former covering work to be carried out during the next five years at a cost of $5,500,000 and the latter covering the probable extension in the ensuing twenty-five years at an approximate cost of another $5,500,000. This plan makes it possible to go ahead immediately with sewer construction wherever needed, and at the same time coordinates the work in various districts so that all is combined into the most advantageous system.

Population Curve

For the purpose of estimating the population increase of Vancouver the growth of six American and three Canadian cities has been plotted, as shown in the accompanying diagram, the starting point in each case being taken at a time when the city had a population of 150,000, which was approximately the population of Burrard peninsula in 1911. On this chart it is noticeable that inland cities show a slower rate of growth than seaports, and also that there is a great difference in rates. Growing at the rate that applied to Boston in the forty-year period following the 150,000 stage, Vancouver would have a population of 500,000 in 1950, while if it grew at Chicago's rate the population in 1950 would be 2,000,000. The curve for Vancouver has been plotted midway between that of Chicago and New York, and the sewer development has been designed so as to permit of its being extended from time to time until there is provision for a population of 1,400,000.

Five Drainage Areas

Burrard peninsula is divided naturally into five drainage areas, which discharge respectively into Burrard Inlet, False Creek, English Bay, Fraser River and Burnaby Lake. Each of these areas was studied as a separate problem in laying out the general scheme, except that the disposal of sewage from each area was considered in the light of its relation to the other districts or the waters there available for dilution. The volume of sea water subject to tidal flow which was adopted as necessary for diluting sewage without nuisance or injury to fish life was taken as 100 times the quantity of sewage involved.



Dispersion in English Bay of Floats Starting from Two Different Points

The average rainfall in Vancouver for the last seven years has been 56 in. per annum. During 1912 careful records kept at New Westminster showed that on only three days did more than ¼ in. of rain fall in an hour. On the other hand, very heavy downpours occur at rare intervals, and drain capacities are required for storms such as the one that occurred in June, 1913, when there was a precipitation of 0.45 in. in ten minutes and 0.9 in. in thirty minutes. The sewerage systems now under way, however, are being constructed on the combined system, with the exception of the False Creek and English Bay areas.

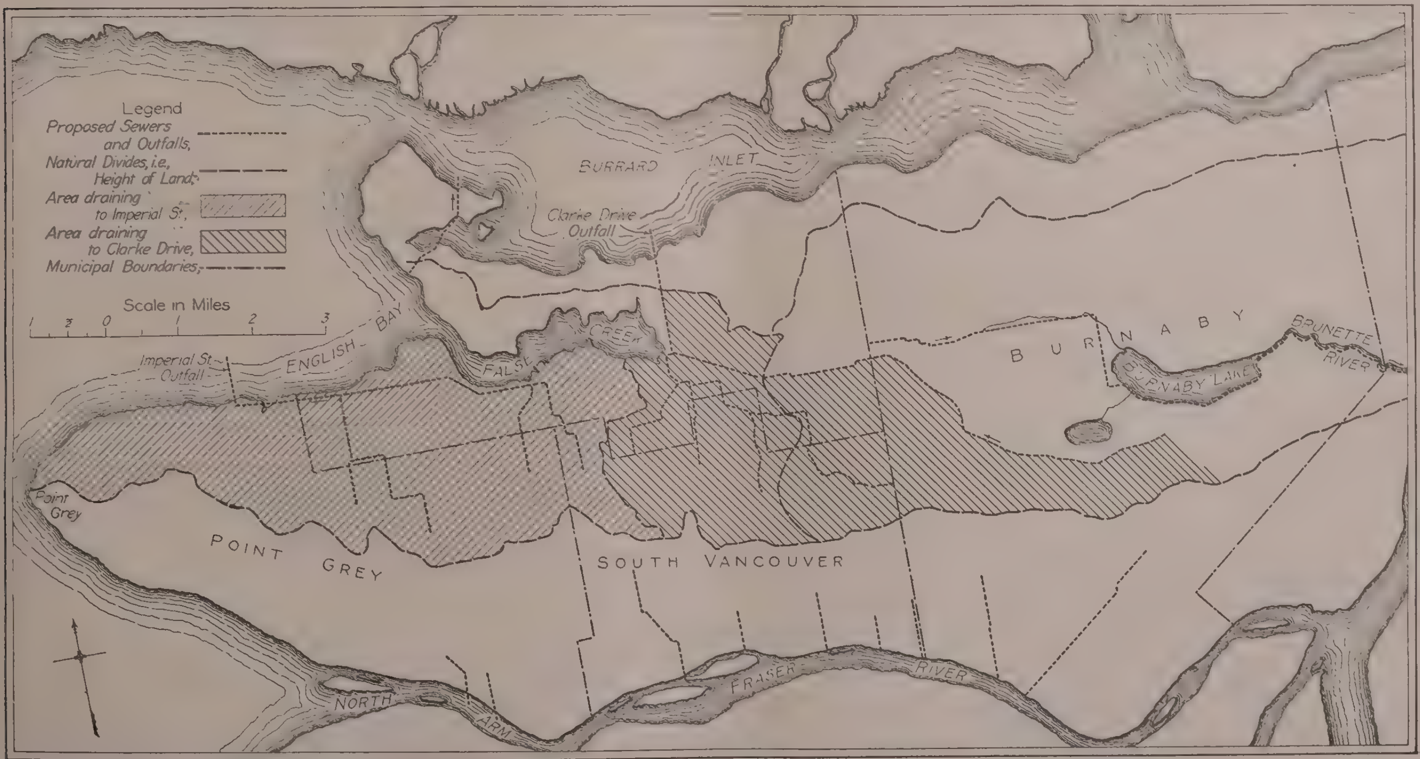
English Bay and False Creek

The total area draining into English Bay and False Creek is 8650 acres, which is estimated to have a population of 270,000 in 1950. The standard of purity demanded for the foreshore waters of English Bay is high owing to the presence of bathing beaches, and for the area affecting these waters a separate sewerage system is advised, limiting the combined system to those parts of this basin where it has already been installed. The point of discharge selected for this area is in English Bay, some 5000 ft. out from the shore on the line of Imperial Street. The first outfall pipe will be 60 in. in diameter, and will be carried out, at first, a distance of 3000 ft. and will be extended and other outfall pipes added as the population increases. One of the reasons for selecting this outfall location was that there is a stretch of level land near by on which some form of sewage-treatment works could be installed, should these become necessary at some future time.

The route to be followed by the interceptor passes first along the high-water mark on a grade of 1 in 2700, the sewer here having an inside diameter of 9 ft. and beginning with its invert elevation 93 ft. above the city datum. At Balaclava Street, meeting the Balaclava sewer, the diameter decreases to 8 ft. and the interceptor continues on a grade of 1 in 2400. Five main trunk sewers feed this interceptor, as well as two low-level areas from which sewage will eventually have to be pumped. The largest of the five contributory areas is served by the Balaclava trunk sewer, 2350 ft. of which has already been completed. This completed section has an inside section 10 ft. 4 in. wide by 8 ft. 3 in. high, and pending the construction of the interceptor has a temporary outfall direct into English Bay.

Float Measurements

Before selecting the location of the Imperial Street outfall numerous float experiments were made to determine the point where the best diffusion could be obtained.



Plan of Burrard Peninsula, Showing Scheme of Joint Sewerage Systems for Greater Vancouver

The accompanying chart shows the average dispersion of floats for various periods of time. The floats used were cut from 6 x 6-in. timber in 2-ft. lengths, and were weighted so as to float upright. It was found that the Fraser River caused a definite current from Point Grey to the First Narrows at all stages of the tide, the effect being more marked during the high stages of the river. Between this line and the shore there was found to be at all times a circular, clockwise movement of the water. The two float charts reproduced herewith show what comparatively slight advantage there would be in placing the outfall at the extreme end of Point Grey, although at first sight this might seem to be by far the more desirable location.

Fresh and Salt Water

An important feature of the Fraser River effect on English Bay is that the fresh water spreads out above the salt water and covers a larger area than would be the case if these waters, by reason of temperature differences, mingled along the line of contact. The Fraser River water normally extends to a depth of 10 ft. below the surface, the depth reducing to a thin sheet along the line of advance of an incoming current. The river water carries much glacial silt and its muddy color readily distinguishes it from the clear water of the bay. This color difference often makes the slight depth of the overlying fresh water apparent in the wake of vessels, a path of clear water marking that which has been churned from below by the propeller. The surface blanket of fresh water will have a beneficial effect on sewage dilution, it is believed, because of its lower specific gravity, which will retard the tendency of sewage to float. Although the prevailing winds at the outfall are off shore, arrangements are to be made for the interception of all floating matter, so as to prevent the possibility of any of it being carried back and strewn along the beach.

False Creek has an area at high tide of about 1½ sq. miles, and the volume entering it on a 10-ft. tide is about 1,600,000,000 gal. Due to the fact that there are bathing beaches on either side of its outlet, it is not considered a suitable place for the dilution of raw sewage. It is, however, looked upon as a relief outlet, in times of heavy storm, for those adjacent areas where the combined system is in use.

Burrard Inlet

The principal discharge into Burrard Inlet will be through an outfall located at Clarke Drive, which is to drain an area of 8250 acres, with a population, it is estimated, of 238,000 in 1950. Parts of the area being made tributary to the Clarke Drive sewer would naturally drain into False Creek and Burnaby Lake. In view of the fact that the upper part of False Creek is about to be filled in, it is recommended that suitable provision be made prior to this fill for a relief outlet, which would come into use should the Clarke Drive outfall become gorged by any extraordinary rainfall.

In rating the capacity of Burrard In'et for diluting sewage it was assumed that at some future date a dam might be thrown across the Second Narrows. For this reason, only that area between First and Second Narrows was considered available, and of this area one-half was figured as belonging to the north shore. Thus, omitting as before the low-water volume, there remained one-half the tidal volume between the First and Second Narrows, which would, on a basis of 1 to 100, effectively dilute the sewage from a population of 1,200,000. Besides the Clarke Drive trunk there are smaller sewers which are to discharge in Burrard Inlet, the largest being the Hastings Park outfall, draining 800 acres. The only requirement for these smaller sewers is that the outfall pipe be carried beyond the pier line into deep water.

Because of there being little through current in Coal Harbor no outfall will be permitted between Brocton Point and the Canadian Pacific Railway wharf. The present sewers discharging into this area are to be intercepted and the sewage carried across to the northeast shore of Stanley Park, where the tidal currents run as high as 9 knots per hour.

Burnaby Lake

Burnaby Lake has a mean water area of about 430 acres, with a maximum depth of 9 ft. After a period of dry weather it is practically stagnant and not capable of digesting any amount of raw sewage. The total area draining into this lake is about 26 sq. miles, which has its only outlet through the Brunette River. By 1950 it is estimated that the population of this watershed will be about 200,000, and the placing of streets and roofs over the present highly absorbent soil of the basin will materially change the runoff. It is estimated that with this population the heaviest storms will give rise to a runoff from the watershed of at least 4000 sec.-ft. That is to say, assuming a velocity of 5 ft. per second and a channel depth of 10 ft., the outlet will have to be 80 ft. wide. It is pointed out that it will be of the utmost importance to conserve the natural drainage channels of this district. In the estimates for "immediate construction" there is an item of $200,000 for improving Still Creek and the Brunette River and providing two interceptors for sewage disposal during the next five years.

Fraser River

The minimum discharge of the Fraser River, occurring in March, amounts to about 20,000 sec.-ft. at Hope, B. C., which is 80 miles above New Westminster. The maximum flow, which comes in June, is about 40,000 sec.-ft. Just below New Westminster the north arm leaves the main river, and the influence of tides here makes it difficult to determine the proportional flow down each of the channels. It is believed, however, that the flow in the north arm will never fall below 1,000,000,000 gal. per day under present conditions. This amount, on the standard basis of dilution, would handle the sewage from 100,000 persons. On the same basis the minimum flow of the river would oxidize the sewage of 900,000 people.

The area contributory to the north arm of the Fraser River includes 17,000 acres, and in 1950 the estimated population on this area will be 250,000. To meet the needs of the district in the next five years the combined system trunk sewers indicated on the map have been provided for in estimates for "Immediate Construction." It is believed that the time will come when an interceptor will have to be constructed or some form of local treatment introduced in this area. Provision for the former has been made in estimates on "Deferred Construction."

The surveys, estimates and designs embodied in the report submitted by Mr. Lea were made under the personal supervision of Mr. A. D. Creer, Mem. Can. Soc. C. E., A. M. I. C. E., who is still in charge of engineering work for the Greater Vancouver joint sewerage board.

Growth of the Cement Industry on Pacific Coast

The cement industry on the Pacific Coast is a little more than twenty-nine years old. Its growth during this period was the subject of a paper delivered by Mr. Charles A. Newall before the Pacific Northwest Society of Engineers. The speaker pointed out that in the late eighties and early nineties only two plants in California were making natural cement from argillaceous limestone. This product, however, was not trustworthy, being exceedingly irregular in quality.

The first plant to make true Portland cement of good quality on the Coast was built in Oregon, and it is stated that it was the first to use a rotary kiln successfully. In 1894 Uria Cummings planned to build a plant near Los Angeles, but as satisfactory transportation rates could not be agreed upon with the railroads the plan was abandoned. In 1898 the first plant to operate successfully was started by the California Portland Cement Company at Cotton, 50 miles east of Los Angeles. Its initial capacity was 500 bbl. per day. The product was poor, and not until four years ago, when the new mill was built to increase the capacity to 2500 bbl. per day, and the plant was remodeled, was a high grade of cement produced.

In 1903 two plants were started—the Standard Portland Cement Company, at Napa Junction, and the Pacific Portland Cement Company, at Cement. Their products, Mr. Newall said, compare favorably with others on the market. The plants have been enlarged from time to time and in 1906 each company built a new mill.

In 1907 the Washington Portland Cement Company began operation at Concrete, in Skagit County, with a capacity of 800 bbl. per day. Three years later the capitalization was raised to $700,000, and the capacity was increased to 2500 bbl. per day. The next year, 1908, the Superior Portland Cement Company was started, the plant being not quite one-half mile away from the plant of the Washington cement company. The Superior is capitalized at $1,000,000, with an output of 1500 bbl. per day. In 1909 two plants were started—the plant of Riverside Portland Cement Company, at Riverside, Cal., with a capacity of 3000 bbl. per day, and the plant owned by the Los Angeles Aqueduct Commission.

In 1910 the plant of the Golden State Portland Cement Company was put in operation at Oro Grande—on the Mojave Desert in southern California. This plant is capitalized at $200,000 and rated at 1000 bbl. per day. In 1911 the Inland Portland Cement Company, a subsidiary of the Lehigh Portland Cement Company, of Allentown, Pa., began operation at Metalane, north of Spokane. In 1913 three new plants will be in operation—the International, just out of Spokane; the Olympic, in Bellingham, and the San Juan, near Watsonville, on Monterey Bay, California.

In the twenty-nine years since 1884 the cement industry on the coast has grown from the single plant at Oregon City, with its 100 bbl. per day capacity and $50,000 capitalization, to a total of thirteen plants capitalized at about $24,000,000 and with a combined output of about 38,500 bbl. per day.

The Waterfalls Merdala and Aura, Romsdalen, in the western part of Norway, will be developed by the Nitrogene Trodents and Carbide Company, of London, at a cost of $10,000,000. The period for which the falls will be leased has not been determined.

Book Reviews

The American Institute of Consulting Engineers has issued a 92-page pamphlet on "The Status and Professional Relations of the Engineer," being the report of discussions held at meetings of the institute on Dec. 18, 1912, and March 11, 1913. These discussions, participated in by the most eminent engineers in New York City, have attracted much attention and were reprinted at length in the Engineering Record last winter. The compiling of the discussions in a handy volume is much to be commended. (American Institute of Consulting Engineers, 101 Park Avenue, New York City.)

The U. S. Geological Survey has issued a pamphlet on the "Ohio Valley Flood of March-April, 1913," as Water Supply Paper 334. The work is by Messrs. A. H. Horton and H. J. Jackson. In addition to discussing this flood it makes comparisons with some of the earlier inundations. Particular attention is given to the floods of March-April, 1907, and of 1884. The paper includes the usual data on causes of the floods, their progress, record stages, the effects of tributaries, etc. There are interesting chapters at the end on the losses caused by the last flood and on prevention of flood damage. (Washington, D. C., U. S. Geological Survey.)

The "Year Book" of the American Society for Testing Materials for 1913, containing the regulations of the society and all of the standard specifications in full, was sent out late last month. Extensive review of the volume is unnecessary here, since its scope is well known. Unlike previous issues of the "Year Book," the list of members is not included, but will be published separately. (Philadelphia, American Society for Testing Materials, Edgar Marburg, Secretary-Treasurer. $5.)

In order that all persons interested in the improvement and maintenance of highways in the State of New York may be kept in touch with the progress of the work the State Highway Commission has issued the "New York Highway News." If practicable, arrangements will be made to issue the bulletin periodically, later editions taking up and discussing local projects in all sections of the State. The first issue has been devoted largely to expressions from the principal executives of the commission, giving in detail the scope of their duties, authorities under which their functions are exercised and policies to be pursued. (Albany, N. Y., State Highway Commission.)

Hydraulic Turbines. By R. L. Dougherty, Assistant Professor, Sibley College, Cornell University. Cloth, 6¼ x 9¼ in.; 156 pages; 96 illustrations. New York, McGraw-Hill Book Company. $2 net.

With increased importance of water-power development has come about the necessity for efficient hydraulic equipment. It is generally the practice of manufacturers of such equipment to recommend to the designing hydraulic engineer what is most suitable in the case in question, but it is also of vital importance for this engineer to be familiar enough with the subject to enable him to check up the recommendations. The manufacturer, of course, is looking out for his own interests, and it has happened that wheels have been recommended because the design and patterns were at hand in the shops, although the speed was not the one suitable under the prevailing conditions.

In this book the author gives a comprehensive description of the various elements governing the design of various types of waterwheels, their construction and the characteristics necessary for an intelligent selection of the type and size of turbine most suitable for any given set of conditions. The book is therefore written not so much for the turbine designer as for the designing hydroelectric engineer, although the clear understanding of the theory ought to be of interest to many turbine designers.

The subject is divided into thirteen main chapters: types of turbines and settings; water power; tangential waterwheel; reaction turbine; general theory; theory of tangential waterwheel; theory of reaction turbine; turbine testing; general laws and constants; selection of a turbine; selection of type of turbine; cost of turbines and waterpower, and centrifugal pumps.

Especially interesting are the chapters dealing with commercial constants, cost of turbines and comparison of cost of water power with steam power. The book will prove valuable to engineers as a reference and to students as a text book.

Transformer Practice. By William T. Taylor. Cloth, 6¼ x 9¼ in.; 278 pages; 190 illustrations. New York, McGraw-Hill Book Company. $2.50 net.

This book, as its title indicates, deals with the practical side of the subject and presupposes on the part of the reader a knowledge of the fundamental principles of electrical engineering. It is of primary interest to those engaged in power-plant construction or operation.

A brief history of the development of transformer construction is followed by a very complete discussion of different methods of transformer connection in single, two, three and six-phase systems. Each case is illustrated with a diagram showing the voltages at various parts of the system and a vector diagram showing the phase relation of electromotive forces and currents.

Of the other subjects treated the most important are: Methods of cooling transformers; properties of transformer oil; construction, installation and operation of large transformers; auto transformers, constant-current and series transformers; regulators and compensators; practical testing of transformers; specifications and records of tests.

Steam: Its Generation and Use. (Thirty-fifth Edition.) Babcock & Wilcox Company. Cloth, 10¼ x 7½ in.; 335 pages; 40 illustrations. New York, Babcock & Wilcox Company.

This book is greatly enlarged from the preceding edition and the treatment of nearly all subjects is more complete. The subject matter is such as to give much general instruction in the generation and use of steam.

There are, besides the forty illustrations cited above, many cuts of Babcock & Wilcox boilers, superheaters and chain-grate stokers showing details of construction, and also much information concerning them, thus enabling the reader to understand the workings of the apparatus.

The book is interspersed with much valuable information on the following general subjects: Early history of steam generation; circulation of water in steam boilers; treatise on feed water; description of properties of steam and air. with tables; combustion of fuels and fuel and gas analysis; furnaces for different fuels; chimney design; boiler testing, care and operation of boilers, and boiler-room piping.

The company is to be commended for producing a book of such merit. The work is worthy of a place in any engineering library.

Technical Gas and Fuel Analysis. By Alfred H. White, Professor of Chemical Engineering, University of Michigan. Cloth, 5¾ x 8¼ in.; 276 pages; 47 illustrations. New York, McGraw-Hill Book Company. $2 net.

The following partial list of subjects treated gives an idea of the work covered: Heating value of gas and candle-power of illuminating gas; chimney and producer gas; liquid fuels; sampling and analysis of coal and determination of its heating value by means of several forms of calorimeter.

Descriptions of apparatus are given in great detail and much attention is directed to methods of calibration and precautions to be taken against leakage of air or gas that might tend to vitiate the experiment. As is very natural, the author pays special attention to the methods used at the University of Michigan and to apparatus that he has found to work well in his own laboratory. Whenever computations are necessary in connection with an analysis an illustrative case is worked out in detail. The work is based on the methods advised by the various technical societies and is well adapted to the needs of the student or of the young chemist who is called upon to do commercial testing without special training in this branch of chemical work.

Overhead Electric-Power Transmission. By Alfred Still, Assistant Professor of Electrical Engineering, Purdue University. Cloth, 6½ x 9¼ in.; 307 pages; 115 illustrations. New York, McGraw-Hill Book Company. $3 net.

Straight, long-distance, overhead transmission of electric energy is one of the vital problems of to-day and few engineers not directly occupied in this particular line realize the number of different points that have to be considered in the design of transmitters. The author of this book has treated in a simple and direct way the fundamental principles and scientific laws determining the design of such conductors. Distribution systems are not considered. The subject is treated less from the standpoint of the construction man than of the office engineer whose duty it is to make the necessary calculations and draw up the specifications.

The book is divided into eight chapters and seven appendices. The following subjects are dealt with: Principles and theory; economic principles and calculations; electrical principles and calculations; insulation and lightning protection; transmission of energy by continuous current; mechanical principles and calculations, and transmission line supports. The appendices treat of inductance of electric-transmission lines with unsymmetrically disposed conductors; inductance of electric transmission lines as affected by subdivision of the circuit and the arrangement of the conductors; two-phase transmission with three conductors; approximate method of determining deflections and stresses in flexible tower lines; graphical statics applied to transmission-line calculations, with special reference to steep grades; sample specification for wood-pole transmission line and sample specification for steel-tower transmission line.

An attempt has been made to give reasons for things in the simplest possible terms and higher mathematics has been avoided. Vector diagrams, supplemented, where necessary, by trigonometrical formulæ, have been freely used in the solution of alternating-current problems. The book will prove useful not only to the practical designer but also to the engineering student specializing in power transmission.

Letters to the Editor

Unfairness to the Contractor

Sir: Apropos of the recent discussion in your journal regarding unfair clauses in contracts and specifications I would call your attention to paragraph 14 of the "General Provisions" of building contracts with the Bureau of Yards and Docks of the Navy Department. It reads as follows:

"14. *Changes.*—The United States reserves the right to make such changes in the contract, plans, and specifications as may be deemed necessary or advisable, and the contractor agrees to proceed with such changes as directed in writing by the chief of the Bureau of Yards and Docks. The cost of said changes shall be estimated by the officer in charge, and if less than $500 shall be ascertained by him. If the cost of said changes is $500 or more, as estimated by the officer in charge, the same shall be ascertained by a board of not less than three officers or other representatives of the United States, to be appointed under the direction of the chief of the Bureau of Yards and Docks. The cost of changes as ascertained above, when approved by the chief of the Bureau of Yards and Docks, shall be added to or deducted from the contract price, and the contractor agrees and consents that the contract price thus increased or decreased shall be accepted in full satisfaction for all work done under the contract: Provided, that the increased cost shall be the estimated actual cost to the contractor at the time of such estimate and that the decreased cost shall be the actual or market value at the time the contract was made, both plus a profit of 10 per cent: And provided further, that changes amounting to $500 or more shall not be made except after being authorized in writing by the Secretary or Assistant Secretary of the Navy, such authority to be cited in the order for the change."

This paragraph, you will note, provides for the adjustment of all changes without consulting the contractor at all, except, of course, if the officer in charge were inclined to be fair. On the other hand, if the officer were inclined to be autocratic he could cause a contractor serious loss on small changes in the work. Even on changes in excess of $500, the amount of which is ascertained by a "board," the contractor could be prevented from having a hearing.

While, of course, it is improbable, it is conceivable that there might be a difference of several thousand dollars between the cost of a change as ascertained by the board and the actual cost to the contractor. Yet under this paragraph the contractor would have to accept the award.

Another paragraph which might work injustice to a contractor is No. 24, reading as follows:

"24. *Order, Protection, and Completion of Work.*—The contractor shall protect his materials and work from deterioration and damage during construction, and upon completion shall without delay remove his plant and all surplus material and rubbish from the site. The contractor will be required to carry on his work without interfering with the ordinary use of the streets or with the operations of other contractors or delaying or hindering any work done by the United States, whether upon the site or not. He shall make good any damage to property of the United States caused by his operations. It is understood and agreed that the parties to the contract will, so far as possible, labor to mutual advantage where their several works in the above-mentioned or in unforeseen instances touch upon or interfere with each other. Mutual concessions under the direction of the officer in charge shall be made to secure this end. It is also further understood and agreed that from any such necessary interference, whether resulting in delay or additional expense or not, no claim for extra compensation shall arise, the contract price covering all contingencies of every kind, except for changes provided for elsewhere in this specification."

It is conceivable that a contractor working under this clause might suffer serious damage where the Government subsequently let another contract which might cause him a long delay in the completion of his own work. On a job involving much plant and overhead expense every delay makes deep inroads into the estimated profits.

Charles F. Dingman,
Secretary, Merrick Fireproofing Company.
New York.

Irrigation of Santa Cruz Valley

Sir: What is to follow is not written in the spirit of a criticism to be applied directly to the Santa Cruz system as published in the Engineering Record of Aug. 23, page 200, and Aug. 30, page 242, but is given as a pointer to engineers and others called upon to investigate and construct systems of this nature. It is to be assumed by one reading the article that the work as shown and completed is a success, is above criticism of a vital nature, and is a plan worthy of being followed in making similar installations. The information given in the article is not sufficient for the writer to judge the merits nor to criticise the installation.

The point to be brought out is this: After drilling investigations have been made across the valley, the wells have been developed and tested, the depth to water measured, the velocity of the flow of the water through the gravels measured by the Slichter method, how efficiently is the water going to be collected by the pumps? Efficiency in this case does not mean economy in the operation of the pump by the saving of power, but it does mean the diversion of enough water at all times for the pumps, even if the water is actually there.

There are certain conditions which influence the flow of water in sands and gravels that are very often overlooked. The engineer accustomed to dealing with water flowing in open channels and who is inexperienced with ground-water problems is very likely to fail in this kind of an installation.

In a well that is being pumped there is a "draw-down" curve, or a lowering of the water surface, as the water approaches the well casing. This is caused by the water having to take an increased velocity as it approaches a central point. The depth of the water surface in the well when the pump is operating will depend upon the amount of water being pumped, and is likely to vary several feet from the normal level.

In a line of wells set across the valley, planned to divert a large percentage of the total flow in the gravel, the fluctuation of the water levels is very irregular. A dam resting on an impervious stratum lying below the gravel bed would raise the ground-water levels to a uniform height except in the direct vicinity of the well. In this case the water would be running over the dam until the pumps diverted the full volume of water flowing down the valley. In the Santa Cruz installation, if we assume that there will be a time when the pumps are diverting the total supply coming down the valley and that there is not a natural submerged dam below the line of pumps, the water table just above the pumps will dip toward the pumps. The water below the pumps will lower by being drained off; the result will be that the pumps will be compelled to operate from a greater depth with a probable decrease in capacity.

In making this kind of an installation the setting of the pumps is a serious matter. A pump set below the surface of the water, for instance 10 ft., cannot pump water when the water has lowered an additional 27 ft. or so. A 6-in. pump in a well with a strainer very often draws the water down 25 ft., partly due to the resistance offered to the water by the strainer and adjacent sand beds. It is thus seen that a comparatively small fluctuation of the normal water surface in a gravel bed will make the pumps useless.

Arthur L. Collins,
San Francisco. Consulting Engineer.

[The effect of the "draw-down" curve on the spacing of the pumps is clearly described by Mr. M. C. Hinderlider, the author of the article above referred to, in the last paragraph of the first column on page 201 of the Aug. 23 issue.—Editor.]

Wilful Water Waste

Sir: With regard to the subject of water-waste prevention which has been discussed in various articles in recent numbers of the Engineering Record, a notice reading, "Important! Please Leave Water Running in All Faucets," was found posted in the bathroom of a large rooming house in the suburbs of the city, and it was obviously the intention of the owner of the property to avoid the freezing of the water in the pipes and faucets by keeping a continuous flow during actual or threatened cold weather. In this way it was possible for him to permit the temperature to fall below freezing without suffering damage. This seems to be a clear instance where people are perfectly willing to cause the public a great deal of expense in order that they may save a very little. It is water waste of this kind which is largely stopped by the installation of meters. The saving of water due to additional care in making repairs and stopping leaks probably amounts to fully as much as the above saving, but the wilful waste usually occurs during cold weather, when the water system is put to the greatest expense in many other ways.

New York City. W. E. Belcher.

New Jersey Sewage-Disposal Plants are classified in a tabulation issued by Mr. R. B. Fitz-Randolph, chief of the Board of Health of that State. There were 144 sewage plants in the State on Aug. 1—95 public and 49 private—all except four of which have been built since 1900. In most cases sedimentation supplemented with contact or trickling beds is used. The greatest average sewage flow is 1,700,000 gal. per day at Asbury Park. Lime is used in the few plants employing a precipitating chemical and calcium chloride is the disinfecting chemical in all but one case, at a hospital in Trenton, where ozone is used. In 44 plants the water from the stream receiving the sewage effluent is subsequently used for drinking purposes. The Board of Health has ordered practically every municipality in the State to construct disposal works.

Oceans Meet at Panama

By the blasting of the Gamboa Dike across the Panama Canal on Oct. 10 the last barrier separating the Atlantic Ocean from the Pacific was removed and the great waterway now extends across the Isthmus, practically completed. The removal of the dike signalizes the termination of dry excavation at Panama and the work remaining to be done consists largely in applying the finishing touches and in installing and "tuning-up" the equipment needed for operating the canal. The remaining excavation in the Culebra Cut will be removed by floating dredges.

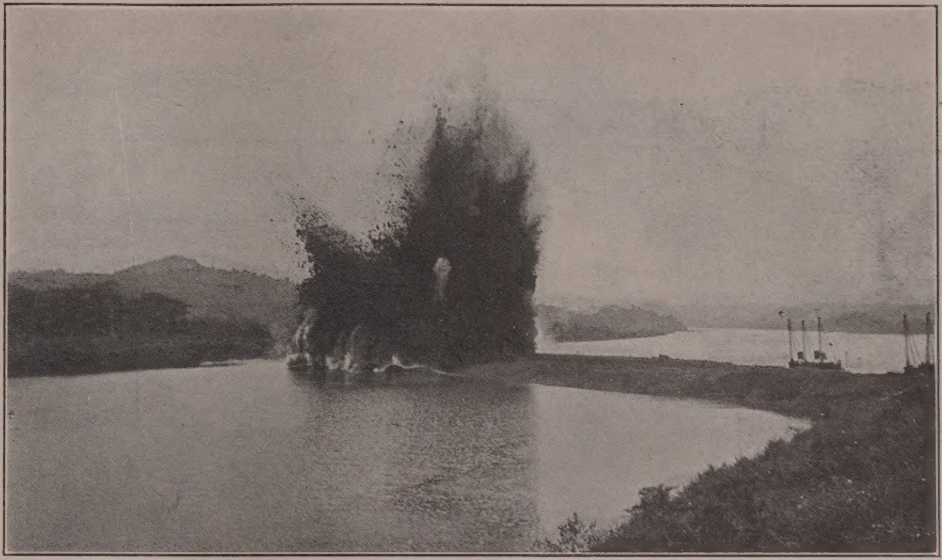


Fig. 1—Blowing Up Gamboa Dike, Last Barrier between Atlantic and Pacific Oceans

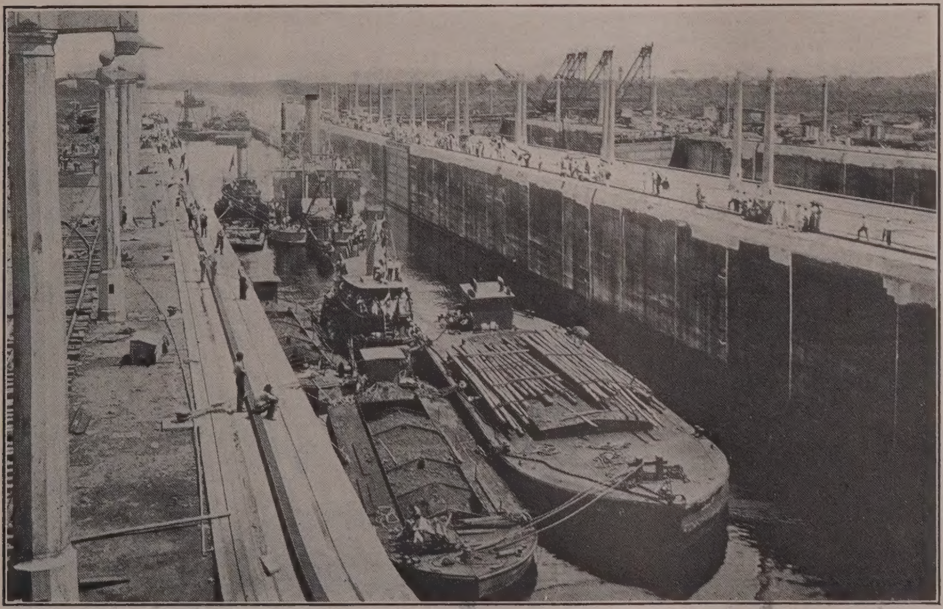


Fig. 2—Gatun Locks Looking North toward Atlantic Entrance of Canal

The six photographs which accompany this article were taken within the last two weeks and show the present state of the Panama Canal work. Fig. 1 is a view of the blast which shattered the Gamboa Dike. Drilling of holes was completed on Wednesday, Oct. 8, and the work of loading them was begun the day before. More than 1000 holes, extending over every part of the barrier, were drilled to an average depth of 35 ft. Water was first admitted into Culebra Cut through four 24-in. pipes leading into the lake underneath Gamboa Dike at 9 a. m. on Wednesday, Oct. 1. It was calculated that with a velocity flow of about 24.1 ft. per second the discharge would amount to 354 cu. ft. per second, which would fill the canal as far as the Cucaracha slide at the rate of 1.6 ft. a day. The slide at Cucaracha completely closed the cut in one place to the 73-ft. level, and therefore the water discharged into the canal by means of the pipes was wholly confined to the section between the slide and the dike. The completion of excavation at the Gamboa Dike will be carried out by a portion of the Atlantic entrance dredging fleet, which were lifted to the level of Lake Gatun through Gatun locks.

Fig. 2 is a view of the Gatun locks looking toward the Atlantic entrance of the canal and shows tugs, dredges and barges ready for the first lockage from sea level up to Lake Gatun. In Fig. 3 are shown the control house at the Miraflores locks and the lock gates ready for the passage of the first ship. The flooded portion of the Culebra Cut between Gamboa Dike and Pedro Miguel locks is shown in Fig. 4. Fig. 5 is a view of the upper locks at Gatun looking toward Lake Gatun. Tugs, dredges and barges are making their first trip through the locks to Lake Gatun. At Pedro Miguel locks there is an emergency dam above the lock gates to protect them. In case of an accident to the gates the emergency dam, shown in Fig. 6, would be swung into position across the lock chamber and a series of wickets lowered across the canal channel to impede the rush of water through a damaged gate.

First Ship through Gatun Locks

The Gatun locks were operated first on Friday, Sept. 26, when the seagoing tug "Gatun" was passed through the west flight from the Atlantic channel to Lake Gatun. The operation was entirely successful and was witnessed by a large number of people.

The first stage in admitting water to the locks was the filling of the space between the upper guard and upper operating gates. This was done through the 60-in. auxiliary culvert valve in the west wall. The operation was started at 9:07 a. m. on Thursday, Sept. 25, and continued to about 3:30 p. m. It was necessary to close the valve twice during this period to clear vegetation away from the

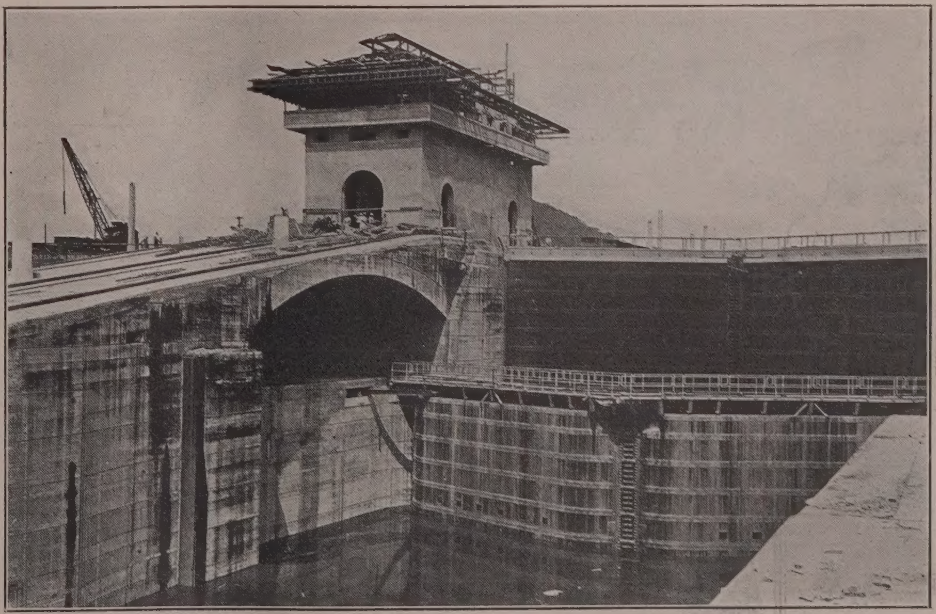


Fig. 3—Control House, Nearly Completed, and Gates at Miraflores Locks

(Photos Copyright, 1913, by Byron Company)

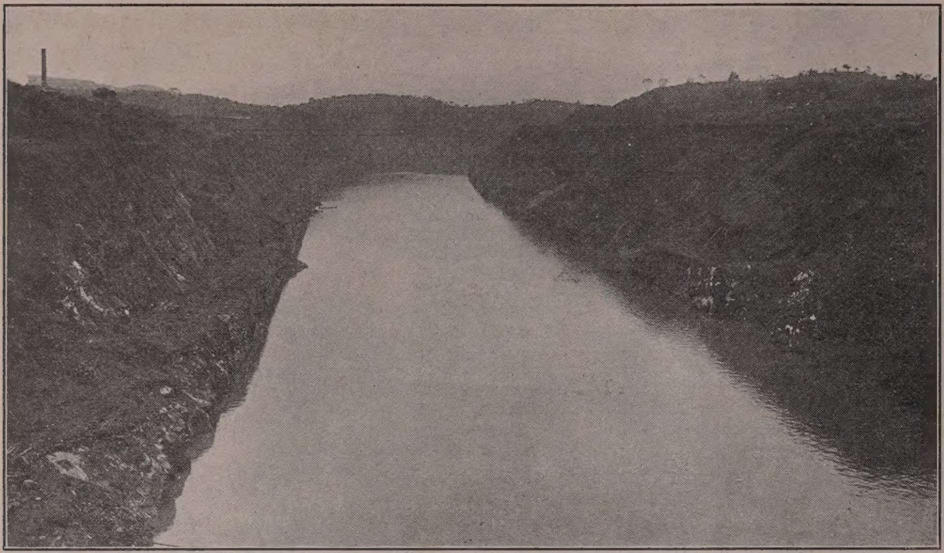


Fig. 4—Flooded Section of Culebra Cut between Gamboa Dike and Pedro Miguel Locks

trash screen at the intake. The second step consisted in filling the culvert in the center wall as a test of the valves and bulkheads as well as the culvert itself. At 10 a. m. on Friday, Sept. 26, the bulkheads were removed from the upper end of the west culvert and water was admitted to the upper lock and brought to lake level. The upper rising stem valves were then closed and water was passed down the flight of three locks as a preliminary test of the valves and culverts of the west walls. Water was then locked down step by step from the lake to the lower lock, which was also being filled by the two 14-in. sea valves in the lower guard gate.

The filling of the lower lock was completed about 4:45 p. m., when the sea gate was opened and the "Gatun," with flags flying and whistles blowing, steamed into the lower lock. The lower operating gates were closed and the tug came to a stop alongside of the center wall, to which it was moored. The process was repeated in the middle of the lock, and at 6:15, just at dusk, the vessel entered the upper lock for the last lift. This was accomplished at 6:45, when the two last gates were swung and the tug passed out into Lake Gatun, the entire process occupying approximately one hour and fifty-one minutes. The filling of the lock chambers occupied a longer time than normally because of the small displacement of the tug. In the case of a larger vessel much less water would have been used. The total lift was approximately 64.7 ft., divided between the three locks as follows: Lower lock, 11.2 ft.; middle lock, 23.7 ft., and upper lock, 29.8 ft.

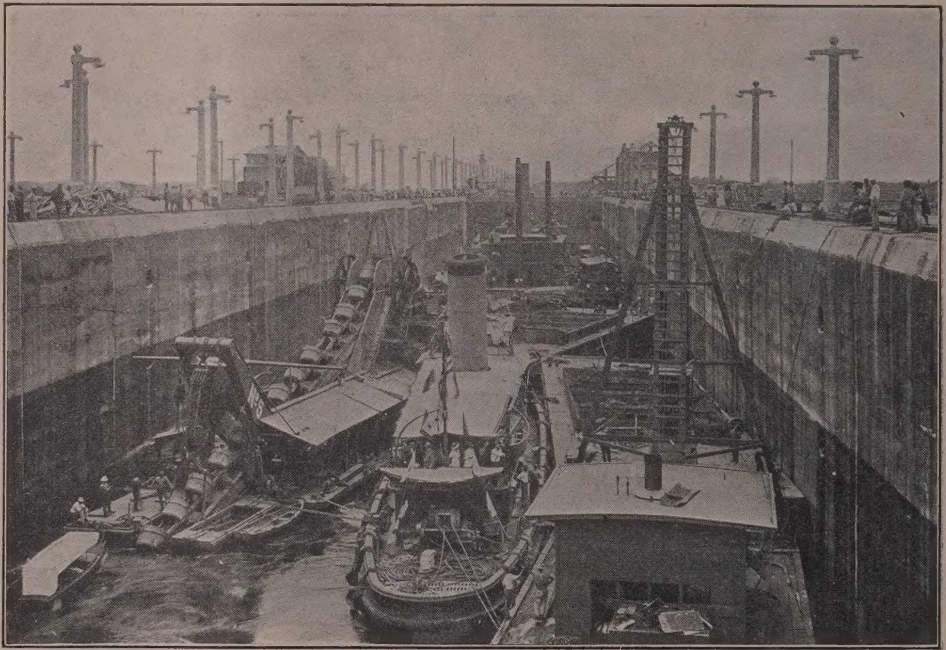


Fig. 5—Tugs and Barges Making First Lockage to Gatun Lake

The opening and closing of the gates and the operation of the culvert valves and lock-operating machinery during the test and the lockage of the vessel were under the immediate supervision of Col. H. F. Hodges, the designer of the locks and their equipment and of the operating machinery and regulating work of the canal. In this he was aided by Mr. Edward Schildhauer, electrical and mechanical engineer of the commission and inventor of the bull-wheel which opens and closes the lock gates; Mr. E. E. Lee, superintendent of erection; Mr. F. S. Clark, assistant superintendent of erection; Mr. C. P. Fortney, superintendent of mechanical installation; Mr. F. H. Grauten, testing engineer, and Mr. George F. Guynn, inspector of lock gates and protective devices.

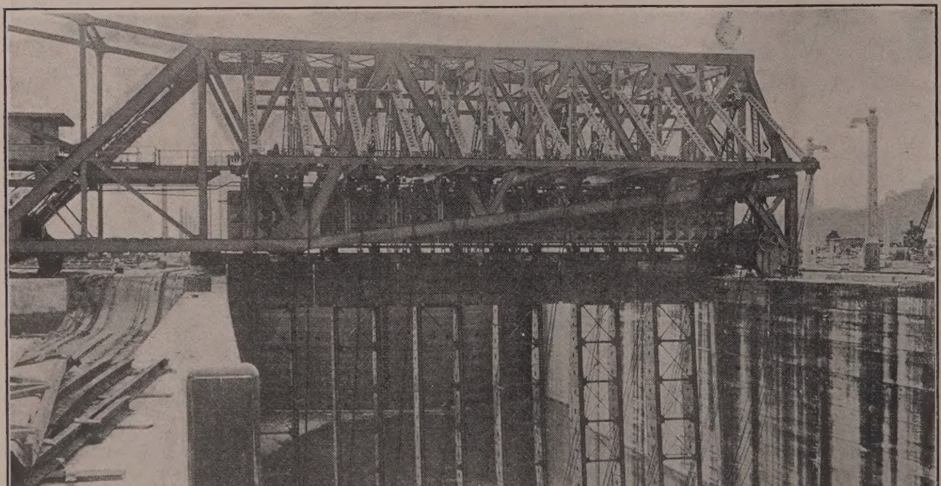


Fig. 6—Emergency Dam above Lock Gates at Pedro Miguel

(Photos Copyright, 1913, by Byron Company)

The "Gatun" was commanded by Capt. F. F. Stewart and its navigation through the locks was in personal charge of Mr. W. G. Comber. The vessel had among its passengers on its initial trip Col. H. F. Hodges, Lieut.-Col. William L. Sibert and family, Maj. James P. Jerzey and Mrs. Jerzey, Maj. George M. Hoffman, Lieut. George R. Goethals and Mrs. Goethals, Mr. Henry Goldmark, Capt. Corning, Mrs. Edward Schildhauer and Mrs. E. E. Lee.

On Saturday morning, Sept. 27, the tug was returned to the Atlantic channel, its progress through these locks occupying one hour and thirty-seven minutes.

With the blowing up of Gamboa Dike on Oct. 10 the Central Division of the canal was abolished. The division, as noted in the Engineering Record last week, was organized on July 1, 1909, and was assigned the canal territory extending from the south toe of Gatun Dam to the north end of the lock site at Pedro Miguel, a distance along the axis of the canal of 31.69 miles. The canal work was divided into five districts and was in charge of Lieut.-Col. D. D. Gaillard, as division engineer.